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SOIL STRUCTURE AND THE STEP-STRAIN PHENOMENON

By D. H. Trollope,<sup>1</sup> M. ASCE and C. K. Chan,<sup>2</sup> A. M. ASCE

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SYNOPSIS

Following recent advances in the field of colloid science, a working hypothesis aimed at describing the make-up of stable soil structures has been developed and is described. In the light of this hypothesis, the mechanism of shear failure in soils and their response to long-term sustained and repeated loads has been studied.

A phenomenon which has been observed in the laboratory and is termed step-strain behavior is reported, and a possible explanation of this phenomenon in terms of soil structure is presented. Finally, an experimental investigation aimed at evaluating some of the predictions of the theory with respect to creep behavior and ultimate strength is described.

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INTRODUCTION

Since the early days of the pioneer work of Karl Terzaghi, Hon. M. ASCE in developing a scientific approach to the study of the engineering behavior of soils, the question of the physico-chemical nature of the soil constituents has attracted many workers in this field. Terzaghi was the first to recognize the importance of these factors, and his work was followed by the well known hypothesis of the structure of marine sediments by A. Casagrande, F. ASCE.

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During recent years a wealth of additional knowledge has accrued from developments in colloid chemistry. This has thrown some light on the behavior of that soil fraction which previously was least understood—the clay colloids. However, the theoretical study of colloid systems has been based on what has become known as the parallel plate model—or the disperse system. Analysis of this model considers electro-chemical forces only, and external forces are ignored.

The hypothesis presented in this paper seeks to include—in a qualitative sense—the effect of external forces, and to develop simplified models of possible stable colloid arrangements as a basis for further study.

### THE SOIL STRUCTURE

During the past decade, workers in the field of soil mechanics have become increasingly aware of the important role played by colloid science in developing an understanding of the fundamental nature of soils. As a result, mainly, of the work of T. William Lambe, F. ASCE, Michaels, et al., of the Soil Stabilization Laboratory, Massachusetts Institute of Technology, the advances gained by investigations of colloidal activity have been adapted and made intelligible to the engineer interested in the shear characteristics of soils. Furthermore, work such as that carried out by I. Th. Rosenqvist, L. Bjerrum, A. W. Skempton and R. Northey has thrown some light on soil-structure phenomena and has stimulated both interest and further study.

At the present time, therefore, it is possible to present a rational description of the physical make-up of soils in broad terms. Of necessity, such a description must be mainly qualitative and contain considerable oversimplification of detailed particle behavior. However, if the resulting model serves only to predict trends of soil behavior it occupies an important place in the theoretical and practical armory of the soil engineer.

In any soil aggregate it has become customary to recognize two distinct particle categories:

1. Granular particles (sand and silt\* size particles); and
2. Colloidal particles (clay size particles).

The principal differences between members of these categories lie in the size and shape of the individual particles and their mineralogical composition.

The granular particles in soils are characterized by their quasi-spherical or subangular shape, and their size is usually such ( $>2\mu$ ) that their ratio of surface area to volume is low. However, there is ample evidence from the behavior of sands in shear that for practical purposes simple friction between surfaces is a sufficiently accurate description of the resistance of such particles to relative deformation.

Because of their small size (less than about  $2\mu$ ) and usually plate-like form, the surface area per unit volume of the colloid particles present in a soil is very high. It has been well established that the seat of colloidal activity lies in the electro-static charge on the surface of such particles and, thus electrical forces (rather than gravitational forces as in the case of granular particles) determine their behavior.

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\* (It is recognized that certain particles classified as "silt-size" from sedimentation analyses may well belong in the clay fraction on account of their flaky nature. However, the author believes that the majority of silt-size particles belong to the quasi-spherical or subangular range under the broad classification of granular particles.)

When, as is most often the case, these particles are in the presence of water, the polar structure of the water molecule results in a strong attraction of a layer of water molecules on the surface of the particle that leads to the concept of the adsorbed water film.

The structural characteristics of this water film vary from those of a highly viscous fluid to those of free water—the viscosity decreasing with distance from the particle surface.

Yet another feature of colloidal behavior which has to be taken into account is the ionic distribution within these layers. Soil water, being in general a dilute electrolyte, carries with it ions of +ve or -ve charge. This leads to the concept of the electric double-layer described in an earlier paper by Lambe.<sup>3a,3b</sup>

Thus in a system comprising a large number of colloid particles, the electrical field and ionic distribution between and around the particles is very complex. In addition, account must be taken of the influence of the long range Van der Waals-London attractive forces on the equilibrium distribution of these particles.

The reader is referred to an excellent summary of the nature of these secondary valence forces by Lambe,<sup>4</sup> and no attempt will be made to detail them in this paper. Here, it is sufficient to draw attention to the fact that as a result of recent work in the field of colloid science it has been found possible to estimate the order of the net force between particles, and it has been concluded that the condition of minimum free energy is attained when colloidal particles are oriented parallel to one another. It is, therefore, axiomatic that any colloidal system will tend to this arrangement which results in a minimum of free energy. On this basis, therefore, a simple parallel-plate arrangement may be used as the model for calculating the net interparticle force. A typical curve expressing the potential energy distribution is shown as Curve I in Fig. 1(b). The net force is repulsive or attractive depending on the distance separating the particles. Curve II, in Fig. 1(b), illustrates a condition which is attained with increasing electrolyte content in the pore water.

There are two basic arrangements of colloidal particles that have been identified—the oriented system and the “cardhouse” system.<sup>5</sup> These arrangements are shown in Fig. 2.

The oriented system corresponds in the ideal case to the simple structure shown in Fig. 1(a) that has been used to compute the full-line potential energy diagrams of Fig. 1(b).

Theoretically, therefore, the interparticle electro-forces may be attractive or repulsive depending on the particle separation. In turn, however, the particle separation is also dependent on external forces.

At the conditions represented by Point A (Fig. 1(b)) the particles are at relatively large distances apart and the system is in equilibrium in the absence of external forces. In this range, separation distances are such that the colloid

<sup>3</sup> “The Structure of Compacted Clay,” by T. W. Lambe, Journal of the Soil Mechanics and Foundations Div., Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May, 1958, pp. 1-34.

<sup>3b</sup> “The Engineering Behaviour of Compacted Clay,” by T. W. Lambe, Journal of the Soil Mechanics and Foundations Div., Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May, 1958, pp. 1-35.

<sup>4</sup> “The Structure of Inorganic Soil,” by T. W. Lambe, Proceedings of the American Society of Civil Engineers, New York, Paper no. 315, 1953, 49 pp.

<sup>5</sup> “Physico-Chemical Properties of Soils: Soil-Water Systems,” by I. Th. Rosenquist, Journal of the Soil Mechanics and Foundation Div., Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 2, April, 1959, pp. 31-53.

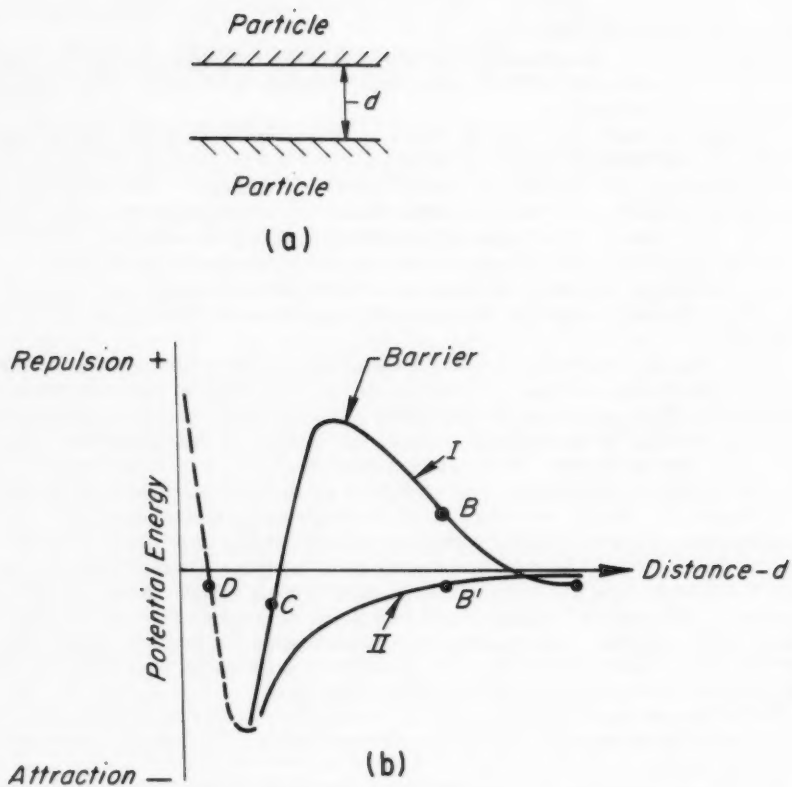


FIG. 1.—ENERGY CURVES FOR IDEALIZED COLLOID MODEL.

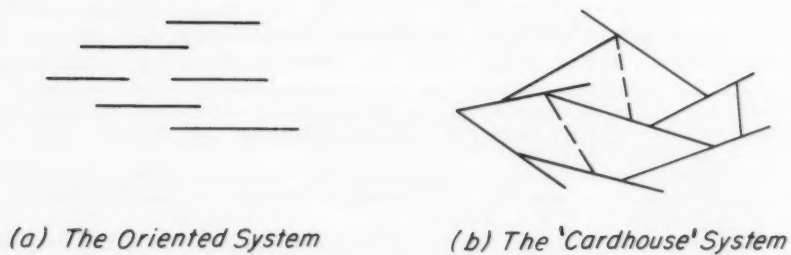


FIG. 2.—TYPICAL COLLOID PARTICLE ARRANGEMENTS.

particles are in dilute suspension corresponding to clay mixtures at water contents well in excess of the liquid limit. If external compressive forces are added then the particles will move into a new equilibrium position represented by Point B. Conversely, to maintain particles at separation distances corresponding to Point B, an external restraint must be maintained. The external restraint may be derived from one or both of two sources; from an externally applied compressive stress (for example, overburden pressure), or by a moisture deficit in the soil, that develops moisture tensions statically equivalent to confining pressures (G. D. Aitchison<sup>6</sup> has shown that effective negative stresses as high as 30 kg per sq cm may be developed in clay soils.)

It has been decided to express the influence of moisture deficit in physical terms, that is, as an equivalent external effective (compressive) stress, rather than the alternative of including it as a variable in the expression for the net potential energy of the disperse system. This alternative implies that an increase in moisture deficit may be considered as an increase in electrolyte content. (In a paper,<sup>3</sup> Lambe has, in fact, used this interpretation as a basis for explaining the types of structure developed in soils compacted at differing degrees of saturation.) This is done on the grounds that the effective stress approach will be of greater significance and use in the engineering sense than a detailed appraisal of the interparticle chemistry which is, in any case, beyond the scope of this paper.

If now the external forces are increased to such an extent that the particles are forced past the 'potential energy barrier' (Fig. 1(b)) then it becomes apparent that conditions at Point C are very unstable and the particles must move into closer proximity, as represented by Point D on the dotted portion of the curve. This latter portion of the curve corresponds to the behavior that has been described by Lambe<sup>3</sup> as "contact interaction." It can be developed either through "contact" of the ordered absorbed water layers surrounding the colloid particles or, more probably, by interaction between these layers and the water shells of hydrated ions, as illustrated in Fig. 3. This interaction is believed to be the source of the significant shearing resistance of clay systems—the magnitude of the resistance being proportional to the repulsive contact forces that are generated.

Clearly in the weak electrolyte environment represented by Curve I (Fig. 1(b)) considerable energy may be entailed in developing this contact, whereas in a strong electrolyte (Curve II, Fig. 1(b)) particles as far apart as those represented by Point B' are in unstable equilibrium and may be expected to move into contact.

This, then, is a possible mechanism for the build up of clay particles, or "packets" as they were originally termed by Terzaghi. Thus it must be appreciated that the colloid particles referred to in this paper may consist of one or many colloid crystals arranged in an oriented system. The term colloid particle is retained as it is inferred that surface activity remains the dominant influence in their behavior.

The 'cardhouse' system is characterized by edge-to-surface "contact" of the plate-like colloid particles. Two factors contribute to the formation of this arrangement. First, the over-all tendency of a mass of plate-like particles deposited at random would be to take up an irregular pattern rather than to immediately fall into an oriented arrangement. Secondly, recent work (J. K.

<sup>6</sup> "The Nature, Extent and Engineering Significance of the Conditions of Unsaturation in Soils Within the Australian Environment," by G. D. Aitchinson, Ph.D. Thesis, Univ. of Melbourne, 1957.

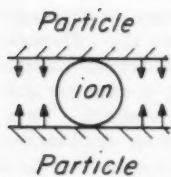


FIG. 3.—DIAGRAMMATIC REPRESENTATION OF PHYSICAL INTERFERENCE OF ION BETWEEN COLLOID PARTICLES.

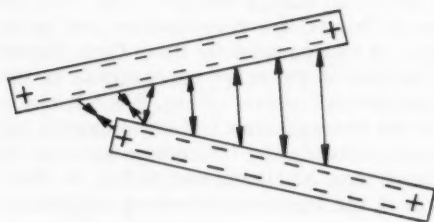


FIG. 4.—EQUILIBRIUM ARRANGEMENT OF IDEALIZED COLLOID MODEL SUBJECT TO ELECTROSTATIC EDGE-SURFACE ATTRACTION.

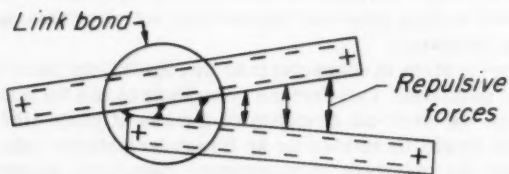


FIG. 5.—FORMATION OF LINK BOND DUE TO EDGE-SURFACE PROXIMITY.

Mitchell<sup>7</sup> and Lambe<sup>3</sup>) has indicated that the natural tendency for edge-surface "contact" is further enhanced by electro-static attraction between negatively charged surfaces and positively charged edges that may be present under certain conditions. It can be readily envisaged, that in the deposition of a sediment, this tendency to edge-surface attraction will be reflected in the formation of the characteristic open framework of the 'cardhouse' system. However, if in addition to this edge-surface attraction there is to be no interplay of surface-surface forces, it must be presupposed that the particles are oriented at right angles to one another. Such a system would be highly unstable, and, hence, it must be visualized that a state of equilibrium can be reached in which the repulsive forces of the adjacent surfaces balance the attractive edge-surface forces, as illustrated diagrammatically in Fig. 4.

In this state the bond between the particles is relatively weak. However, with the system predisposed to this arrangement, the application of any external forces will tend to reduce the gap between the edge of one particle and the surface of the other. Clearly, such a situation is highly susceptible to force (stress) concentrations of this nature, and it can be postulated that through these concentrations enough energy is available to "push" the nearer portions of both particles past the "barrier". Thus we arrive at the condition shown in Fig. 5, where the attractive forces joining edge-surface are derived both from negative-positive charge attraction and the interplay of the attraction corresponding to the particle separation of Point C (Fig. 1(b)). It is most likely that in this condition the Van der Waals-London forces of attraction are the major influence, and the contribution of the positive edge to negative surface attraction is to predispose the system to the arrangement wherefrom a much stronger edge-surface bond can be developed. The term "link bond" may be adopted to describe this zone of strong attractive forces, as shown in Fig. 5. In these circumstances it is not a necessary condition of equilibrium that the attractive forces be resisted by physical interference of adsorbed ions. The repulsive forces on the particle provide the required restraint.

It is, of course, most likely that in a typical soil mass a condition of equilibrium will be attained with the influence of external forces added to the interparticle forces. So we arrive at the simplified general model of Fig. 6(a). Here the distribution of interparticle forces along the face of either particle can be represented as in Fig. 6(b). It should here be re-emphasized that the external forces contributing to this state of equilibrium can be derived either from applied stresses or from induced stresses.

This, then, is the first of the so-called 'cardhouse' structure models. It is postulated that this type of structure will be developed in the clay-fraction of soils in which the pore water is a weak electrolyte. Such soils will exhibit marked swelling characteristics on the relief of applied or induced stresses (for example, rebound in a consolidation test).

The second of the 'cardhouse' structure arrangements concerns that which is established in a strong electrolyte environment. It has already been pointed out that under these conditions the net interparticle force is likely to be everywhere attractive. We may, again, infer that the edge-surface attraction will pre-dispose the arrangement to that of an open framework, as shown in Fig. 7. In such a system it must be realized that the disposition of Particle P (Fig. 7) is essential, not incidental, to the stability of the system. The compressive strength of this particle provides the reaction whereby the arrangement can

<sup>7</sup> "The Fabric of Natural Clays and its Relation to Engineering Properties," by J. K. Mitchell, Proceedings, Highway Research Board, Vol. 35, 1956, pp. 693-713.

achieve stability, and, indeed, it can be visualized that a high degree of stability and, hence, a high order of "structure" is so obtained.

It is in this characteristic that the two flocculated systems under discussion have their greatest difference. The first system tends to expand under the influence of internal forces and is now referred to as the diamond (expansive) system, while the second tends to contract under these forces and is called the triangular (contractive) system.

For example, referring to Fig. 2(b), it can readily be visualized that, under the influence of external forces, a diamond arrangement can exist in stable equi-

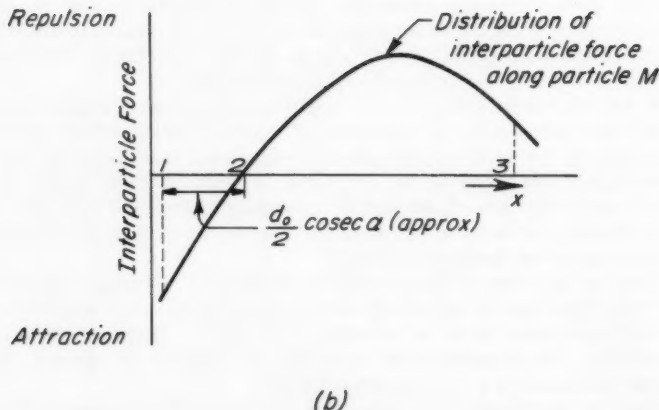
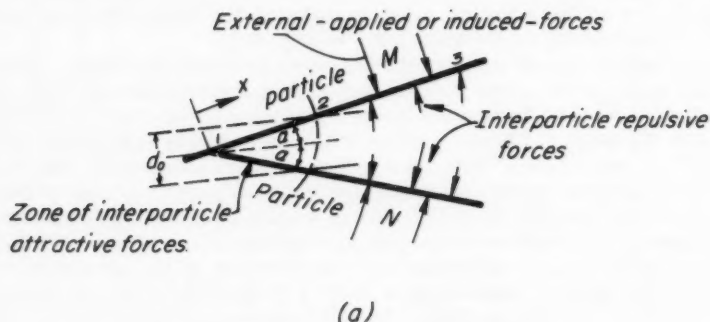


FIG. 6.—ENERGY CURVE FOR IDEALIZED COLLOID MODEL SUBJECTED TO EDGE-SURFACE CONTACT.

librium without the presence of the particles shown dotted in this illustration, whereas these latter particles are essential to the stability of the triangular system, which would otherwise tend to collapse into an oriented arrangement. It may, therefore, be inferred that the former arrangement is much more dependent on external forces in the development of structural strength than is the triangular system.

Yet another aspect of importance in general soil behavior is the reversible nature of these interparticle forces as a result of cation exchange. Consider a soil structure built up of units, as shown in Fig. 7, within a strong electrolyte environment (for example, a marine sediment). It has been suggested previously that, owing to interparticle attraction, such a structure would be highly stable. If now the ions, which under this system neutralize the interparticle repulsive forces, are progressively leached out, the interparticle attraction is progressively reduced and repulsive forces dominate. In this way the system can be changed from a contractive to an expansive state. Then the repulsive forces will tend to separate the points of edge-surface "contact" and in so doing will weaken the link bonds.

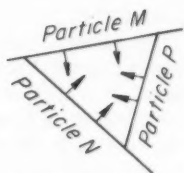


FIG. 7.—THE TRIANGULAR SYSTEM, ALL COLLOID PARTICLES MUTUALLY ATTRACTIVE.

So far, this change from a contractive to an expansive state has involved stress changes of a purely hydrostatic type, and, hence, it is considered that the associated volume change will be very small—this being due to elastic movements of the particles only—as shear stresses are necessary to cause significant volume changes of the type of structure envisaged. If now, however, external loads are applied which do impart shear stresses to the mass, the weakened link-bonds will be easily disrupted and the resultant collapse of the soil structure will generate high pore water pressures.

The mechanism is put forward as a possible interpretation of the phenomenon of "quick" clay behavior, as discussed by Skempton and Northey<sup>8</sup> and L. Bjerrum and Rosenquist.<sup>9</sup>

A corollary of this argument, which could be of some practical interest, is that whereas a diamond structure may be transformed into a triangular structure through an increase in electrolyte content or the application of subfailure shear stresses, there appears to be no mechanism whereby a triangular structure can be transformed into a diamond structure without first developing an oriented structure; that is, without first achieving full yield of the colloid structure.

*Heterogeneous Structure.*—Up to the present, the discussion has centered on the micro-structure of the granular and colloidal materials, which together comprise a typical soil mass. Before proceeding to consider the generalized structure of a heterogeneous soil mass, it is convenient to summarize and identify five possible stable units as shown in Fig. 8. Of these five unit-structures, perhaps the only one which needs further description at this stage is that classed as Type (e)—the loose granular structure stabilized by a colloidal matrix. This type of structure has been previously identified<sup>10</sup> and differs from the loose granular structure, which is classed as unstable and is not considered further in this paper. The effect of the clay matrix is to act as a "void filler" and thus resist the tendency of the larger grains to achieve a

<sup>8</sup> "The Sensitivity of Clays," by A. W. Skempton and R. Northey, *Geotechnique*, Vol. 3, No. 1, 1952, pp. 30-53.

<sup>9</sup> "Some Experiments with Artificially Sedimented Clays," by L. Bjerrum and I. Th. Rosenquist, *Geotechnique*, Vol. 6, No. 3, 1956, pp. 124-136.

<sup>10</sup> "A Study of Sand and Sand: Clay Mixtures in Triaxial Compression," by D. H. Trollope, M. Zafar, *Proceedings, Second Australia-New Zealand Conf. on Soil Mechanics and Foundation Engrg.*, 1956, pp. 7-14.

closer state of packing corresponding to the interlocked granular structure. This concept of a loose granular structure surrounded by a clay matrix is important in the consideration of the mechanism of shear failure which will be developed later.

For the general soil model, that is, that in which both coarse (sand and silt) grains and colloidal (clay) particles are intermixed, no better description can be offered than that previously suggested by Lambe<sup>4</sup> wherein there is a random distribution of coarse grained particles in a clay matrix, Fig. 9.

Some comment should be made here on the nature and degree of remolding. Complete remolding of a colloid structure may be defined as the condition where a 'cardhouse' structure is transformed entirely into an oriented one. In a typical soil, movement of the coarse (sand, silt) grains will be associated with the remolding process.

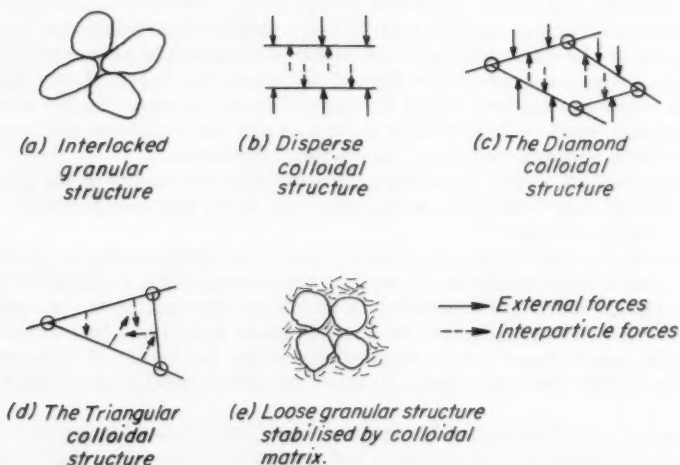


FIG. 8.—FIVE STABLE UNITS OF SOIL STRUCTURE.

Thus, complete remolding will occur only if large shear strains are developed—that is, it will occur on failure surfaces. Compacted soils, which are frequently referred to as remolded soils are generally, therefore, in terms of the above definitions, only partially remolded. It is to be expected that the compacted soil structure will consist of coarse (sand, silt) grains distributed at random in a colloidal matrix consisting of zones of oriented structure surrounded by zones of 'cardhouse' structure. Hence, when a compacted soil is subjected to shear strains sufficient to cause failure, complete remolding will develop along the resulting failure surface.

The definition of yield of a colloidal structure is, therefore, synonymous with complete remolding on a failure surface; that is, yield occurs when an oriented structure is developed along a continuous surface through the soil mass.

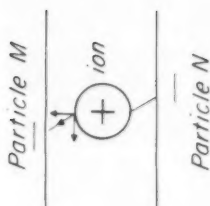


FIG. 10.—SIMPLIFIED MODEL OF IONIC EQUILIBRIUM.

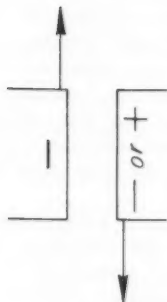


FIG. 11.—MAGNETIC ANALOGY.

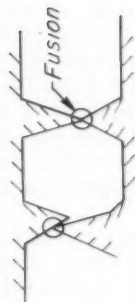


FIG. 12.—FUSION OF ASPERITY CONTACTS, SOLID BODY FRICTION.

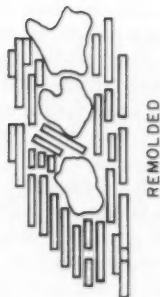
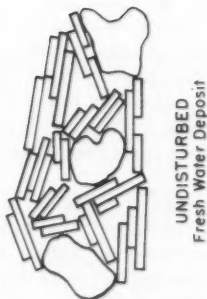
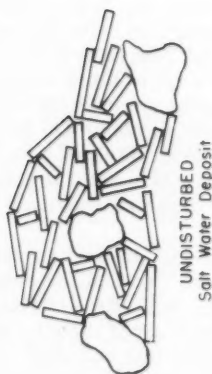


FIG. 9.—UNDISTURBED AND REMOLDED SOIL STRUCTURE.

## THE NATURE OF CLAY "STRENGTH"

Since the inception of soil-strength studies, that part of shear resistance which has been attributed to the clay fraction has, in one form or another, been called cohesion. Over the years the definition of "true cohesion" has changed considerably. At the present time, however, the concept of true cohesion and true angle of internal friction as defined, for example, by Skempton and Bishop,<sup>11</sup> following Hvorslev's earlier work, is being widely adopted and there are strong grounds for doing so. However, confusion is undoubtedly developing because, to the colloid scientist, "cohesion" denotes the strong attractive forces between colloidal particles, whereas to the engineer, "cohesion" is the force per unit area required to displace these particles through large shear strains.

A possible way of avoiding the impending impasse may be derived as follows (see Fig. 10):

Consider a single ion in equilibrium between two particles. This ion occupies its position by virtue of the existence of an electro-static field. It may, therefore, be likened to a ball connected to each plate by an elastic rod. In order to cause shear displacement, particle M has to move laterally relative to particle N, and in the model this cannot be achieved without extending or compressing the rods; that is, work has to be done to achieve this movement. A measure of this work is the horizontal component of the stress in the rod—this is the engineer's 'cohesion.' The colloid scientists' "cohesion," by analogy, is the vertical component of this stress.

From Fig. 10, it is not a big step to the visualizing of shear resistance, as that work which has to be done to displace electric charges (electrons, polar molecules, etc.) within an electric field, and the analogy of the field between two magnets suggest itself, that is, to displace two like or unlike poles of a pair of magnets laterally (Fig. 11) work has to be done in cutting across lines of force.

In the case of intergranular friction a similar mechanism can be developed. When two such "solid" surfaces approach each other, contact is made through asperities as shown in Fig. 12. At these points the stress concentrations are so high that "fusion" results. To cause relative displacement of these asperities, work has to be done to disrupt this fused zone. Again, we have the concept of electrical charges (atoms) having to be displaced in an electrical field. The forces which bond these atoms in the fused zone are most likely of a higher order than the forces operating between colloidal particles in natural soils; however, this does not invalidate the general similarity of the mechanisms in the two cases.

For this reason it is suggested that the term "cohesion," in the soil mechanics sense, be replaced by the term "colloidal friction," so that the shear strength of a soil may be described as shear strength = colloidal friction + intergranular friction. An obvious corollary to this argument is that the strength of a soil is derived entirely from friction, and separation into colloidal and intergranular constituents is unwarranted. It is believed necessary to separate them at the present stage, however, because even if the mechanisms are similar, the order of forces involved differs, and these differences are reflected in the fact that the clay fraction behaves differently in an engineering

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<sup>11</sup> "Building Materials—Their Elasticity and Inelasticity," by A. W. Skempton, A. W. Bishop, Ed. Review, North-Holland Publishing Co., 1954.

sense from the coarser fraction—particularly with respect to the time variable where pore pressure variation is concerned.

### THE STEP-STRAIN PHENOMENON

During a series of slow, incremental loading, undrained tests (to be described subsequently) on a remolded silty clay, it was observed that as the load was built up in equal increments the deflection or strain increment for each loading followed an erratic pattern. This irregularity was manifest both in the total amount of movement and the rate at which the movement developed. Fig. 13 shows the most marked example of this behavior yet observed.

The term "step-strain" has been introduced to describe this phenomenon, since it shows up as a series of irregular steps in the stress-strain diagram and, also, as a step behavior in rate-of-strain studies.

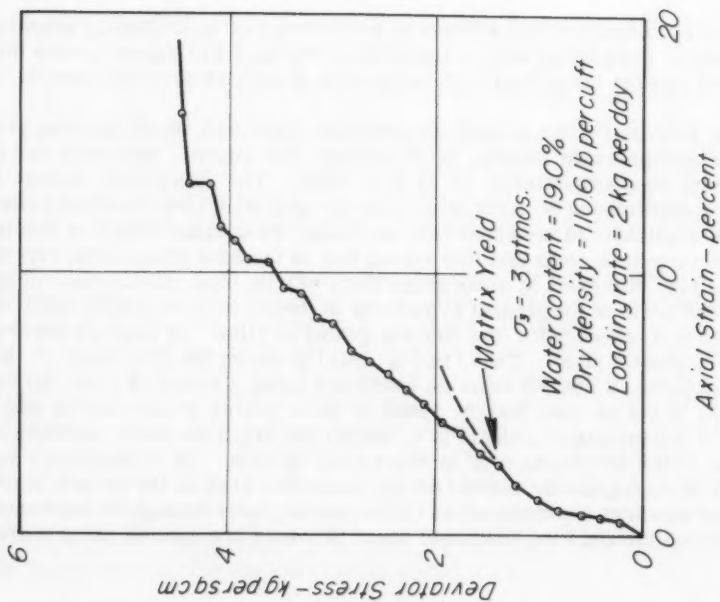
The following general hypothesis has been developed in an attempt to provide a logical explanation for this type of soil behavior. It is recognized that the explanation presented can only furnish a general qualitative description, and further work is necessary before the detailed mechanism can be evaluated.

In the previous part of this paper the most likely arrangements of particles, which go to make up a typical soil structure, have been described. From this the concept of a heterogeneous soil mixture, composed of (coarse) sand and/or silt grains "floating" in a matrix of (colloidal) clay particles is derived.

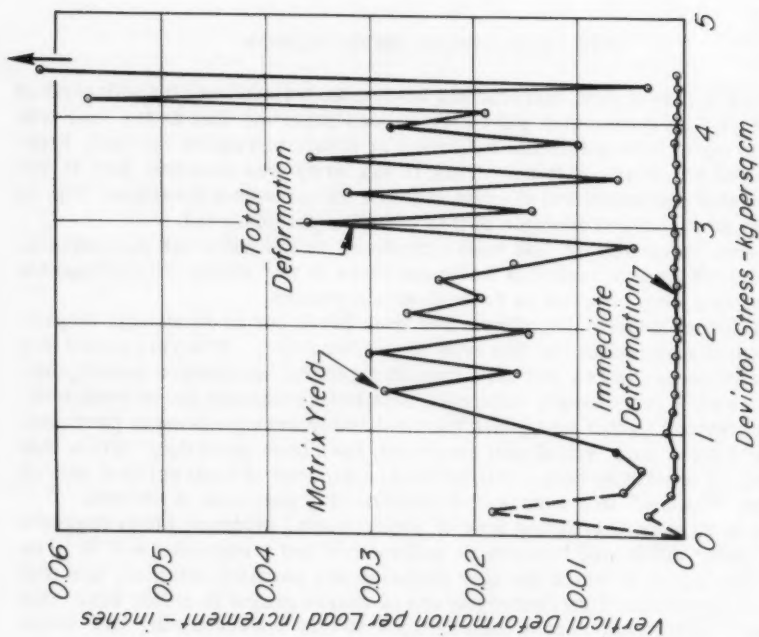
There is an ever-increasing fund of experimental evidence which suggests that the major difference between an undisturbed and a remolded soil is to be found in the degree to which the clay particles are parallel-oriented, and that there is no significant distribution pattern of coarse grains in either case. One of the most significant recent contributions to this understanding has come from the work of Mitchell<sup>7</sup> in his study of thin sections, using light extinction methods.

It appears, therefore, that whether an undisturbed or an artificially prepared (for example, compacted) soil is considered, the model of coarse grains distributed at random throughout, and, in general, separated by, a clay matrix, is valid.

If, now, a cross section through a cylindrical specimen, which has been prepared for compression testing, is visualized, the internal structure can be represented diagrammatically as in Fig. 14(a). The cylindrical sample is chosen for convenience of description only; the argument to be developed would be equally applicable to any shear test specimen. When shear stress is applied to the specimen it is clear that this stress has, in the first place, to be carried by the matrix. However, at some stage there will be local plastic yield of the matrix. The basis of the present hypothesis is that as soon as plastic yield occurs there is a tendency for the coarser grains to "flow" or migrate towards the zone of plastic yield. Thus, in Fig. 14(b) is shown the first stage of this yielding process. If now we examine conditions along a potential shear failure plane X-X, it can be seen that the result of these coarse grains moving into a position of intergranular contact is to add to the available shear strength on this plane by the development of intergranular friction. (It is postulated that the points of intergranular contact occupy negligible area on the failure plane, and applied stresses are transmitted to the coarse grains through the surrounding, unyielded matrix.) As additional shear stresses are applied, progressive



(a) Stress vs Strain Relationship



(b) Axial Deformations for each Load Increment

FIG. 13.—STEP-STRAIN BEHAVIOR IN CONTROLLED RATE OF LOAD TESTS AT CONSTANT VOLUME, VICKSBURG SILTY CLAY.

yield of the matrix occurs with the consequent flow of more and more granular particles into the yielding zone. Ultimately, a condition will be reached where the maximum number of granular particles which can be accommodated within the failure zone will have been fitted in. This ultimate condition is represented in Fig. 14(c), and at this stage it will be seen that the available shear strength on the plane of failure is made up of:

- (a) viscous resistance of clay matrix under plastic yield, and
- (b) intergranular friction from developed granular structure.

It is perhaps necessary to emphasize here that the component of strength (b) in the above expression is not available at the start of the test, or shearing

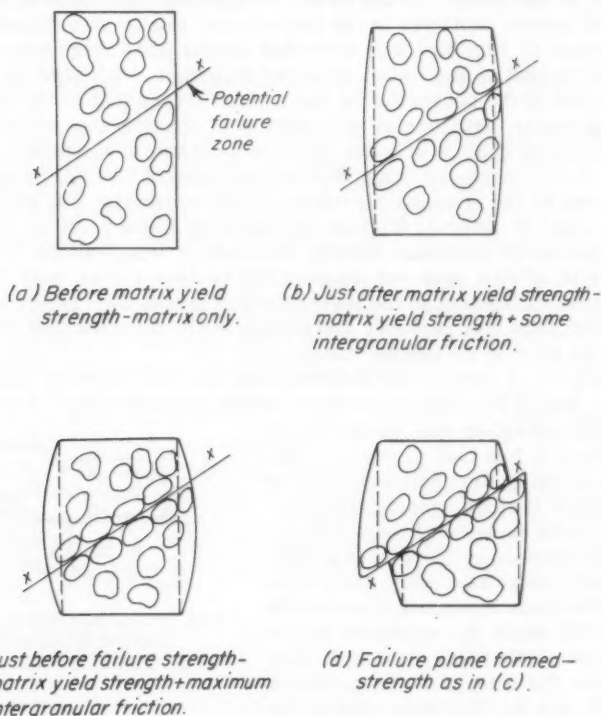


FIG. 14.—DIAGRAMMATIC REPRESENTATION OF BUILD UP OF GRANULAR STRUCTURE AT FAILURE.

process, because the granular structure has to be built up during this process. This concept also provides a likely explanation for the fact that in all attempts to define a shear strength law for soils in terms of "true cohesion" and "true angle of internal friction", it has been found<sup>11</sup> necessary to define these parameters at the void ratio of failure, on the plane of failure; and, furthermore, it has been found that, within the usual void ratio limits for typical clay-type soils, the angle of internal friction is essentially constant.

The step-strain hypothesis provides a ready interpretation of these experimentally observed facts, in that the void-ratio at failure (or water content at failure) for saturated soils is an indirect measure of the particle separation in the yielded matrix and, further, that the granular structure developed at failure is likely to be the same under similar stressing conditions).

The final stage in this model of shear behavior is reached when, a failure plane having been formed, there is distinct movement of one part of the specimen relative to the other. This is illustrated in Fig. 14(d), where it will be seen that the total available intergranular friction can be maintained under considerable relative movement through the mechanism of particles "falling out" of the failure zone at one end and others being admitted at the other end (or anywhere in the zone). In any case, the optimum condition of the maximum number of coarse particles in the failure zone can be maintained in this way.

Reference to Fig. 8(e) will show that in this final stage the soil structure envisaged is that of the loose, granular structure stabilized by clay matrix. The question of the proportion of the applied stress that is carried by the developed granular structure at this point is of considerable scientific interest. As the result of drained triaxial tests on sand:kaolin mixtures (Trollope and Zafar<sup>10</sup>) it was concluded that under the conditions of test, all the applied stress was carried by the granular structure. This conclusion was reached because the true angle of internal friction measured in these tests on the same sand (without kaolin) at minimum density. However, it was realized at the time that this state of affairs may not necessarily be true for all soil mixtures; the maximum kaolin content in the tests referred to was 20% by dry weight of total sample, and, thus, was probably just enough to fill the voids that would be present with the sand in its loosest state.

At the present time, it is believed that the load carried by the granular structure during the failure process varies considerably. For instance, in Fig. 15 the situation can be envisaged where Grain A has just arrived in contact with Grain B; immediately there is a tendency for the load causing shear deformation to be accepted through this intergranular contact as it provides a stiffening effect. Simultaneously, there is a tendency for Grain A to move towards the left; this will build up resistance in the clay matrix on that side, which in turn will relieve the intergranular pressure between A and B. The next step is for Grain A to move into contact with Grain C, and the procedure is repeated.

Ultimately, when complete collapse occurs, it is visualized that intergranular friction is developed through continuous movement within a viscous fluid.

To this mechanism is attributed the fact that under equal increments of load, measured deformations of some test specimens vary widely. Under a given load increment sufficient granular particles may be brought into contact to provide a reserve of strength, so that when the next load increment is applied the already established structure will adequately sustain it without large deformations. However, when a further increment is applied the available shear strength is inadequate, and this last increment cannot be sustained unless additional granular particles are brought into contact through relatively large

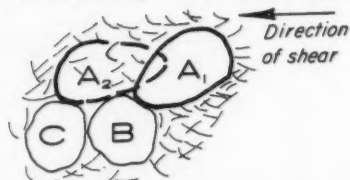


FIG. 15.—MOVEMENT OF LARGE GRAINS WITHIN YIELDED COLLOID MATRIX.

movements, and so on until the maximum available shear strength is reached. Thus, the irregular pattern of the incremental deformation versus stress curve in Fig. 13(b) is accounted for.

Also, within a given load increment the movement described in Fig. 15 is irregular with time. As Grain A moves from position 1 to position 2, under the influence of additional shear stresses, the transfer of pressure is accomplished relatively slowly and with little deformation until the local matrix yield strength is reached; then movement to position 2 occurs relatively quickly until contact with C is established and the rate of deformation is greatly reduced again. Thus, if a graph of deformation vs time is plotted for a given load increment, the relatively sudden movement from position 1 to position 2 will appear as a "step" in the curve. Evidence of this occurrence is shown in Fig. 23 to be presented subsequently).

#### THE EFFECT OF CLAY-STRUCTURE ON STEP-STRAIN BEHAVIOR

In the previous development of the step-strain hypothesis it has been tacitly assumed that the clay matrix does not suffer a significant loss of strength after plastic yield; that is, the clay is of low structural order.

If, however, there is a significant loss of matrix strength at yield, then the conditions necessary for the development of the step-strain phenomenon are not attained.

The general soil model shown in Fig. 9 infers that for yield of a 'cardhouse' clay structure to occur, the link-bonds are destroyed and there is then a tendency for the particles to arrange themselves in an oriented system. Through this behavior the net available shear strength of the locally remolded matrix is reduced.

For step-strain behavior to occur, this loss of matrix strength must be exceeded by the gain in strength through the development of the granular structure. Or conversely, if the maximum potential intergranular strength on the plane of failure is less than the loss of strength of the matrix following yield (due to local remolding) then the step-strain phenomenon will not contribute to the ultimate strength of the soil, and brittle failure will occur. The range of stress-strain diagrams likely to be found in clay type soils is illustrated in Fig. 16. In Fig. 16

Curve I can be obtained with a "pure" clay (little or no coarse particles) of low structural order or with a sand-silt-clay mixture in which the speed of testing obscures the step-strain behavior.

Curve II can be obtained with sand-silt-clay mixtures in which the matrix is of low structural order.

Curve III can be obtained with sand-silt-clay mixtures in which the matrix is of medium structural order.

Curve IV can be obtained with sand-silt-clay mixtures in which the matrix is of high structural order or with "pure" clay of high structural order.

In this connection yet another factor must be considered which plays a significant part in the development of soil structure—moisture tension. It has already been pointed out that moisture tensions equivalent to very high ambient pressures can be developed in clay soils and so contribute markedly to shear strength. A given clay soil which at one water content has the stress-strain characteristics of Curve I, in Fig. 16, can develop brittle failure characteristics (Curve IV, Fig. 16) merely by a reduction in water content.

Apart from the contribution of moisture tension in causing a reduction in interparticle spacing and thus increasing strength, there is the possibility that in unsaturated soils the breakdown of air-water films entails the expenditure of a significant proportion of the shear strain energy, and, thus, when yield is achieved this part of the shear strength is lost and brittle failure characteristics are further accentuated.

Thus, the structural character of the clay matrix plays an important role in determining the nature of the shear failure mechanism in soils. It has been suggested by Mitchell<sup>7</sup> that "since silt particles do not touch each other (in an unsheared soil) but 'float' in a matrix of clay, they probably have little influence on the strength properties of the material." The above argument indicates, however, that this statement should be restricted to cover the maximum shear resistance of soils having clay matrices of medium-high structural order or sensitivity (Curves III and IV, Fig. 16). Where the matrix is of low structural

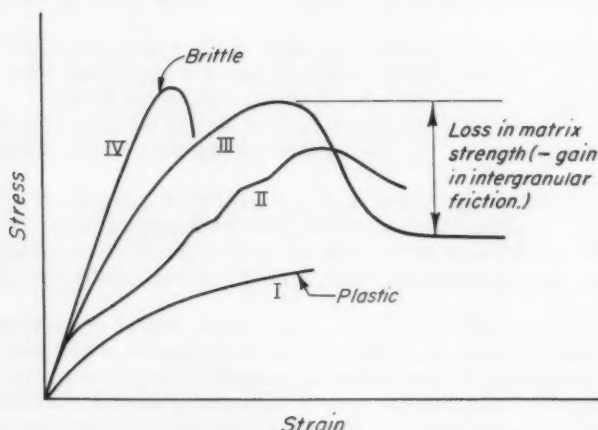


FIG. 16.—TYPICAL STRESS-STRAIN CURVES FOR SOILS OF DIFFERING MATRIX CHARACTERISTICS.

order, then the silt and coarser particles will contribute significantly to shearing strength as they are brought into contact during the shearing process.

#### SCOPE OF EXPERIMENTAL INVESTIGATION

A necessary corollary to the formation of any hypothesis is the setting up of an experimental program that will test the validity of certain predictions made on the basis of the hypothesis. In this context a series of tests was designed to investigate some features of soil behavior which appeared to be of importance and which could be carried out with established techniques.

As the step-strain phenomenon had been observed in tests on a silty clay (Vicksburg silty clay), and this behavior had been attributed to the build up of granular structure following matrix yield, it appeared desirable to carry out comparative tests on samples made entirely from matrix (clay fraction) material. Thus, Wyoming bentonite, supplied in a finely-divided commercial form,

was selected as the material most likely to consist entirely of clay-size particles and hence most likely to have no significant granular structure.

It was also decided that the most convenient method of investigating the features outlined below was by means of undrained, unconfined compression tests on samples prepared at as high a degree of saturation as practicable.

The aim was to eliminate, as far as possible, significant strength variation owing to volume changes, whether due to the expulsion of air or water. With so-called undrained tests made on specimens at high degrees of saturation, it is contended that any air present will compress under relatively light loading and thereafter the pore fluid will be effectively incompressible, thus satisfying the theoretical requirement of the constant volume test.

Three aspects of soil behavior formed the basis of the experimental investigation:

1. The step-strain behavior shows to best advantage in undrained tests in which the load is applied in equal increments with relatively long-time intervals between load applications. Hence, such tests should show a marked difference in behavior between the silty clay on the one hand and bentonite on the other, both in respect to the deformation per load increment and the rate at which such deformation develops. The bentonite should show a general increase in deflection per load increment as the load increased up to the yield point, and also there should be no "step" in the deformation versus time curve within any load increment.

2. Casagrande and S. D. Wilson have described<sup>12</sup> an investigation of creep behavior in which they developed the creep-strength test. In this test a known percentage (say 70%) of the maximum stress measured in a normal undrained test (10-15 min to failure) is applied immediately to the soil specimen, and it is found that for certain soils failure will develop under this lower stress if the load is maintained for considerable periods. The apparent loss of strength measured in this test is referred to as the creep-strength loss of the soil. From consideration of the step-strain hypothesis it appeared that this apparent loss of strength could well be associated with structural changes within the failure zone. For such changes to be significant (for example, causing large, local volume changes with consequent high, local pore pressures), it is most likely that a condition of matrix yield would have to be developed. Thus it was postulated that a soil which was known to exhibit step-strain behavior—the silty clay—would also be more likely to show significant creep-strength loss than a soil—the bentonite—which derived its strength from matrix characteristics only.

Accordingly, it was decided to carry out creep-strength tests on these materials, by applying known percentages of the normal unconfined compressive strength to similar specimens and then maintaining these constant stress conditions for long periods. Such tests have been called "sustained load creep-strength tests" and are described subsequently.

3. In the course of investigations of the behavior of soils, under repeated loads, at the Soil Mechanics Laboratory, University of California, a technique has been developed whereby samples in triaxial-compression can be subjected to repeated applications of a known deviator stress. The importance of this load-simulating mechanism, particularly in relation to road behavior, is very

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<sup>12</sup> "Effect of Rate of Loading on the Strength of Clays and Shales at Constant Water Content," by A. Casagrande and S. D. Wilson, *Geotechnique*, Vol. 2, No. 3, 1950, pp. 251-263

obvious, and the equipment has been developed to the stage where close automatic control of both frequency and duration of load application is possible without incurring impact effects.<sup>13</sup>

As this apparatus was readily available, it was decided to carry out creep-strength tests using repeated, in place of sustained, loads. These tests have been called "repeated load creep-strength tests" and a description of the method is given subsequently.

In addition to providing comparable information with the sustained load tests, it was anticipated that this investigation could possibly throw some light on the sensitivity of certain soils to repeated load applications.

It has been shown<sup>13</sup> for the silty clay that a repeated stress of a given magnitude will produce more axial deformation of a triaxial sample than a sustained load of the same magnitude acting over the same period of time. This is true even though, during this period, the repeated stress acts only for a fraction of the time that the sustained stress acts.

With the type of particle movement described under the step-strain hypothesis, a possible explanation of this behavior can be derived. If a condition is established wherein the granular particles are moving in a "viscous fluid", then this type of structure is liable to be much more sensitive to the rapid repetition of a load than to the same load applied continuously. Therefore, if the hypothesis is soundly based, there should be a much greater difference between the deformation of similar samples, under sustained and repeated loading of the same magnitude, in the case of the silty clay than with the bentonite.

### TEST PROCEDURES

The soils used in the investigation were:

Vicksburg silty clay - Plastic Limit 23, Liquid Limit 37

Wyoming bentonite - Plastic Limit 50, Liquid Limit 400

All tests were performed on specimens having a diameter of 1.4 in. and a height of about 3.6 in. These specimens were prepared in a standard mold using a Harvard miniature compacting apparatus. For the silty clay the preparation technique had been previously developed<sup>13</sup> and reproducible samples could be obtained.

In the case of the bentonite, a number of trial mixes at varying water contents were prepared and the most satisfactory water content selected by inspection. Details of the dry density, moisture contents and degree of saturation attained are given in Figs. 17 and 18.

After preparation, the samples were placed between lucite caps and sealed by means of two thin rubber membranes, with a layer of grease between the membranes. The samples were then stored for 2 to 3 weeks under water. In this way it was hoped to eliminate significant effects of thixotropic strength gain on the results of the long term tests.

To check the extent to which such strength gain with time of storage had occurred, normal undrained tests were performed on control specimens, both

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<sup>13</sup> "Effects of Repeated Loading on the Strength and Deformation of Compacted Clay," by H. B. Seed, C. K. Chan and C. L. Monismith, Proceedings, Highway Research Board, Vol. 34, 1955, pp. 541-558.

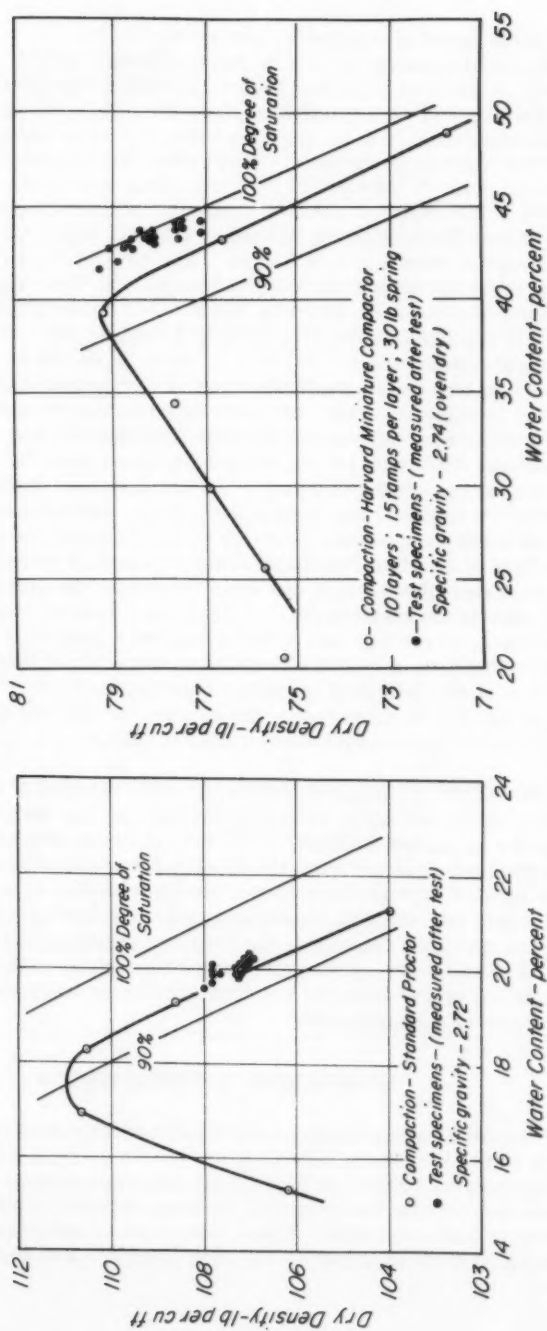


FIG. 17.—COMPOSITION OF TEST SPECIMENS, VICKSBURG SILTY CLAY.

FIG. 18.—COMPOSITION OF TEST SPECIMENS, WYOMING BENTONITE.

immediately after preparation and just before commencement of the long term tests.

The three types of test performed were:

*Incremental Loading Tests (Unconfined Compression).*—The specimens to be tested in this way were assembled in triaxial compression cells and surrounded with water at atmospheric pressure. This was done to maintain constant moisture content in the samples under test. The expected strength of each sample was taken to be the same as the normal unconfined compression strength determined prior to the start of the tests. Each load increment was then taken as 1/10th of the expected ultimate strength, and for any given test the time interval between the application of load increments was kept constant. This time interval varied from 1 min to 2 days. During each load increment measurements of the axial deformation of the sample with time were recorded.

*Sustained-Load Creep-Strength Tests.*—For these tests the samples were installed in triaxial compression cells in a manner similar to that used for the incremental-loading tests. The aim in these tests was to study the creep behavior of samples under constant stress at constant volume. Thus the load to be applied to a given sample was calculated as the desired percentage of the normal unconfined compressive strength. The full load was then applied, quickly but without impact, by letting the weights down on to the carrier of the loading cradle, using a hydraulic jack. As the specimen deformed, further loads were added as required to maintain the constant stress condition. The required additional loads were calculated on the assumption that the specimens deformed as cylinders at constant volume, and, hence, constant stress was only approximately achieved. However, it was considered that the error involved would be insignificant in the test results. As soon as a failure plane was observed in any specimen, no further load corrections were applied, it being assumed that under this condition of relative translation there was a tendency for the stress to increase on the failure plane under constant load.

On completion of these tests, the samples which had not failed were each subjected to an incremental-loading test to failure at a standard rate of 4 kg per min.

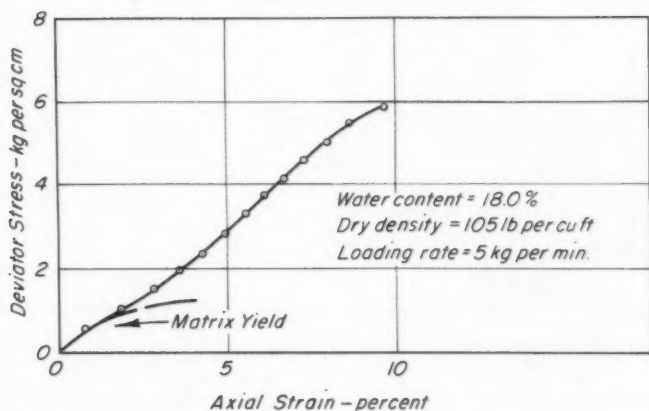
*Repeated-Load Creep-Strength Tests.*—The samples to be tested in this way were assembled in triaxial compression cells as for the previous tests.

As in the sustained load tests, the load to be applied was calculated at the desired percentage of the normal unconfined compressive strength. This load was then applied through the repeated load mechanism at a frequency of 20 applications per min and with each load application having a duration of 0.2 sec.

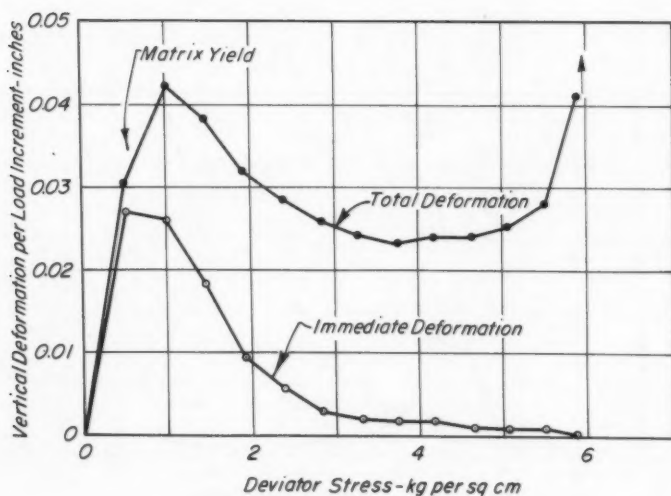
As each specimen deformed, the load was progressively increased so that a constant deviator stress was maintained throughout each experiment. Again, on completion of the tests the unfailed specimens were loaded to failure at a standard rate of 4 kg per min.

## DISCUSSION OF TEST RESULTS

*Incremental Loading Tests.*—The results of four typical incremental loading tests on the Vicksburg Silty Clay are shown in Figs. 13, 19, 20 and 21; with the exception of that shown in Fig. 13, all data were for unconfined compression tests; the test results shown in Fig. 13 were obtained using an ambient stress equivalent to 3 atmospheres. These results are representative of about forty tests on which such information has been observed and analyzed.

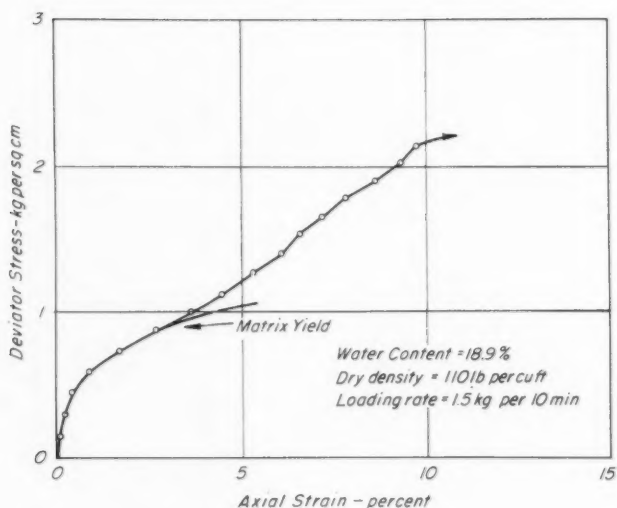


(a) Stress vs Strain Relationship

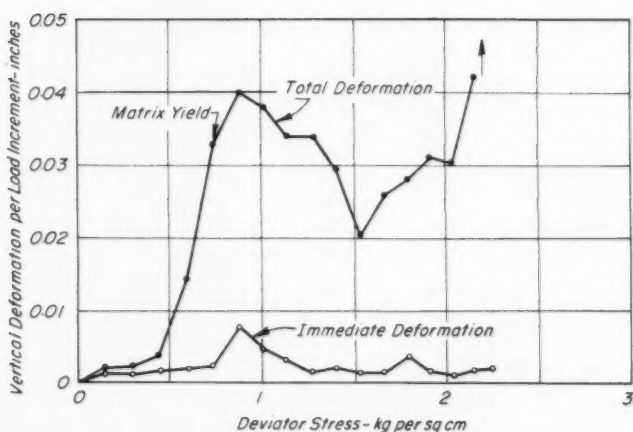


(b) Axial Deformations for each Load Increment

FIG. 19.—CONTROLLED RATE OF LOAD TEST AT CONSTANT VOLUME, VICKSBURG SILTY CLAY.

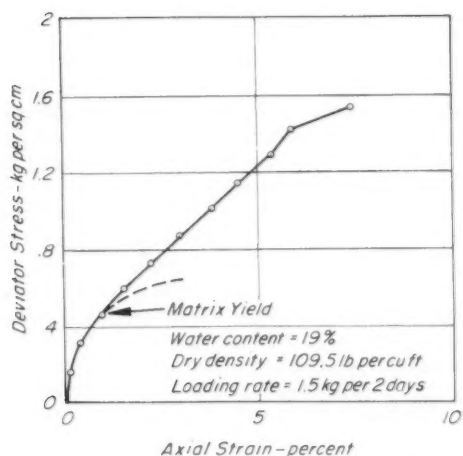


(a) Stress vs Strain Relationship

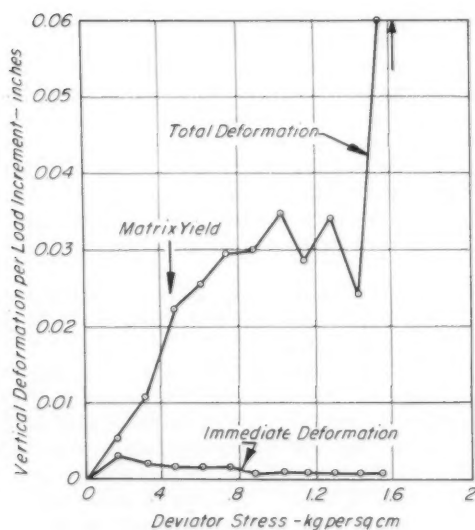


(b) Axial Deformations for each Load Increment.

FIG. 20.—CONTROLLED RATE OF LOAD TEST AT CONSTANT VOLUME, VICKSBURG SILTY CLAY.



(a) Stress vs Strain Relationship



(b) Axial Deformations for each Load Increment

FIG. 21.—CONTROLLED RATE OF LOAD TEST AT CONSTANT VOLUME, VICKSBURG SILTY CLAY.

In Fig. 22, a typical result of the same type of test carried out on Wyoming bentonite is shown.

It will be noted that for the bentonite, the characteristic shape of the incremental deflection curve (Fig. 22(b)) is concave upwards. According to the step-strain hypothesis this type of curve defines the behavior of colloidal material (the clay matrix) under shear strain. The initial peak in the curve can be discounted as being due to initial compression (perhaps the expulsion of small amounts of air) of the specimen.

In Figs. 13 and 19 through 21 it will be seen that portions of these diagrams can be identified as having similar shapes to that of Fig. 22(b), again discounting any initial peaks which may be attributed to initial compression of the specimens. On this basis, therefore, the point of matrix yield for these specimens can be delineated. After a condition of matrix yield has been reached in any specimen, the incremental deformation curve follows a more or less erratic pattern until complete collapse of the soil structure (failure) is attained. This erratic behavior is interpreted as the progressive build-up of the granular structure described under the step-strain hypothesis and is reflected as a series of irregular "steps" in the stress-strain diagram.

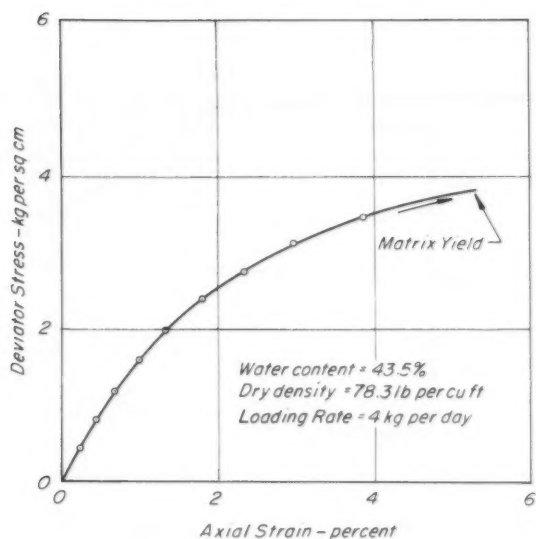
Where the initial behavior of a specimen is erratic, for example, Fig. 13(b), there may be some confusion in attempting to determine the point of matrix yield. However, there is an alternative method of identifying this condition.

In Figs. 23 and 24 are shown typical deformative versus log. time curves for this specimen (LE 13) and for the specimen from which the diagrams of Fig. 22 were obtained for the bentonite. For the bentonite all the curves have the same general shape on the semi-logarithmic plot.

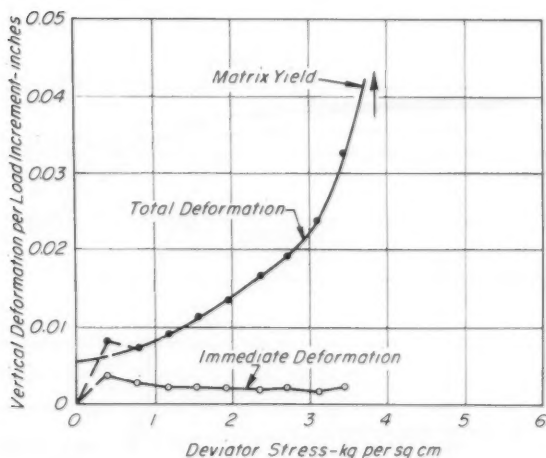
For the silty clay however, the curves for the first few increments have the same general shape as the bentonite curves, but at some stage a different type of deformation curve appears which on the semi-log plot shows as a step. Again it is postulated that this latter curve represents the condition in which, with the matrix under the yield condition, the application of a load increment destroys the equilibrium of one "granular structure" which then proceeds to undergo sufficient movement to develop another "granular structure" which has sufficient strength to resist the increase in shear stress caused by the load increment.

Thus, the onset of matrix yield can be identified where the transition from Curve 4 to Curve 5 (Fig. 23) occurs, and this enables any initial irregularities in sample behavior to be eliminated.

*Creep-Strength Tests.*—The results of the creep-strength tests under sustained and repeated loads for the silty clay are shown in Figs. 25 and 26 and for the bentonite in Figs. 27 and 28. It will be noted that the lowest sustained load under which the silty-clay suffered creep failure was 60%. Since the test specimen under 70% load did not fail, repeat tests were carried out at loads corresponding to 60% and 70% loads. Neither of these specimens failed, and in the case of the 70% load the deformation followed that of the previous specimen so closely that it cannot be differentiated on the diagram of Fig. 27. If creep-strength loss is defined as the difference (in percentage of the immediate (15 min) strength) between the immediate strength and the lowest load under which creep-failure occurs, then it will be seen that for the silty clay, at the moisture content and dry density used in this investigation, the creep-strength loss under sustained loads is 20% to 30%.



(a) Stress vs Strain Relationship



(b) Axial Deformations for each Load Increment.

FIG. 22.—CONTROLLED RATE OF LOAD TEST AT CONSTANT VOLUME, WYOMING BENTONITE.

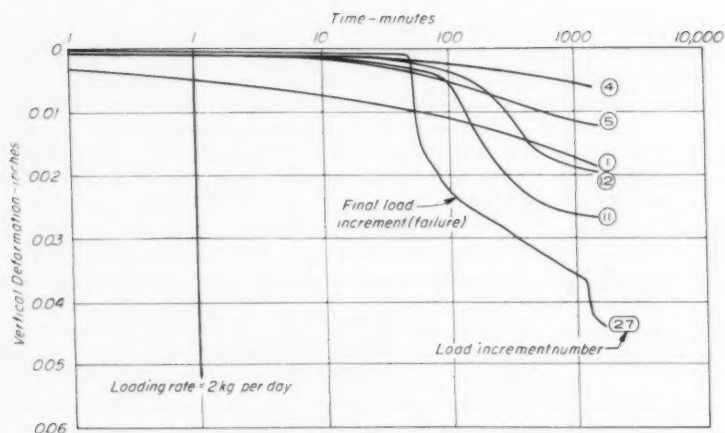


FIG. 23.—TYPICAL DEFORMATION VS TIME CURVES FOR LOAD INCREMENTS IN CONSTANT RATE OF LOAD TESTS ON SILTY CLAY.

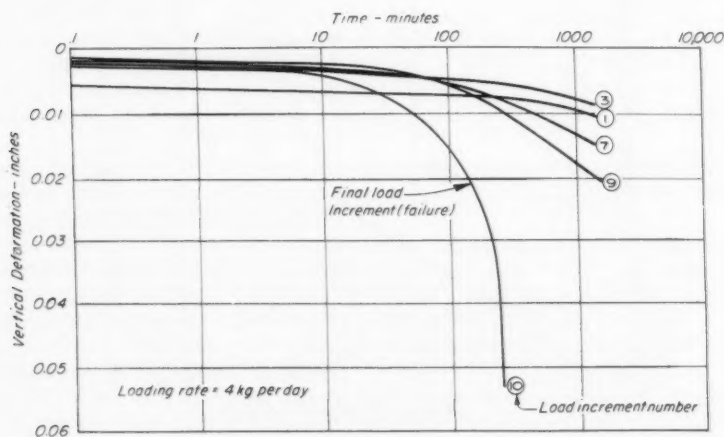


FIG. 24.—TYPICAL DEFORMATION VS TIME CURVES FOR LOAD INCREMENTS IN CONSTANT RATE OF LOAD TESTS ON WYOMING BENONITE.

Similarly, for repeated loading tests, one sample failed under the 80% load, but its manner of failure was so markedly different from that observed in previous tests that this particular result was suspected as being due to some unaccounted-for variation in the sample. A repeat test under 80% load confirmed this, and, hence, it was concluded that the creep-strength loss under repeated loads is 10% to 20% for the silty clay in this condition.

A common feature of the behavior under sustained and repeated loads is the appearance of step-strain behavior in the samples which failed. As soon as a

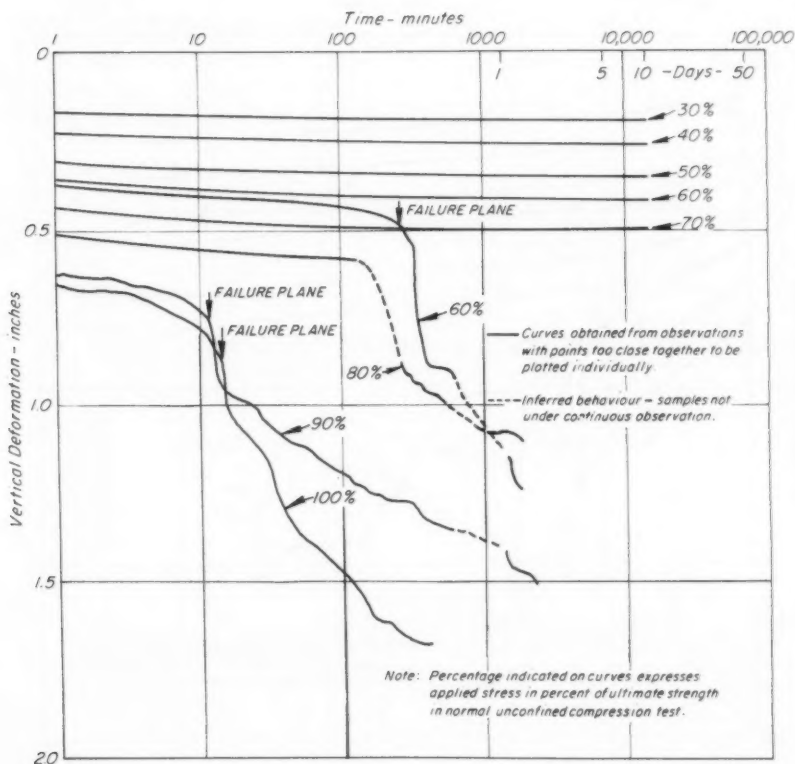


FIG. 25.—DEFORMATION VS TIME CURVES IN SUSTAINED LOAD CREEP STRENGTH TESTS ON SILTY CLAY.

failure plane formed the subsequent deformation versus time curves show the characteristic "steps."

The most significant conclusions that can be drawn from the tests on the samples that did not fail are:

1. The rate of creep is relatively independent of the magnitude of the applied load.
2. Under sustained loads creep is approximately proportional to the logarithm of time.

3. Under repeated loads, creep is only proportional to the logarithm of time (number of repetitions) after an initial period during which creep deformation develops at a much faster rate.

4. Under repeated loads both the magnitude and rate of creep are greater than those for samples under corresponding sustained loads.

These conclusions are interpreted as lending further support to the idea that the soil structure concerned is one of a granular skeleton being deformed in a

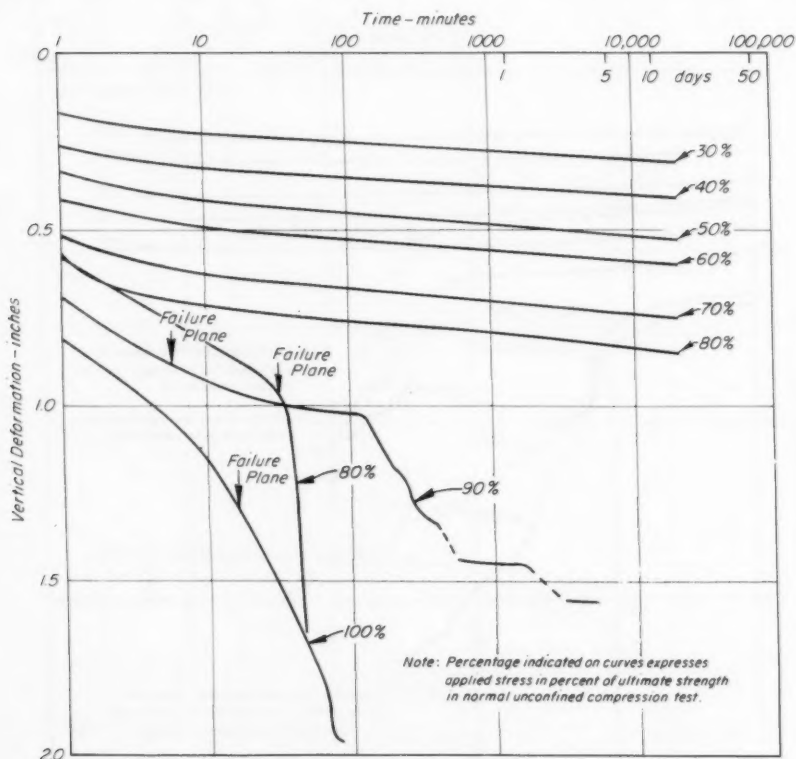


FIG. 26.—DEFORMATION VS TIME CURVES IN REPEATED LOAD CREEP STRENGTH TESTS ON SILTY CLAY.

viscous fluid (yielded matrix). In particular, Conclusion 4 indicates that the analogous "hammer blow" effect of the repeated load application develops more deformation than the "steady push" of the sustained load, and, therefore, in this condition the soil may be classed as repeated-load-sensitive.

By comparison, Figs. 27 and 28 show that for the bentonite at the test condition there is no creep-strength loss under either sustained or repeated loads; and when failure occurs there is no indication of step behavior in the deformation versus time curves. Furthermore, there is no significant difference between the creep deformation developed under repeated and sustained loads

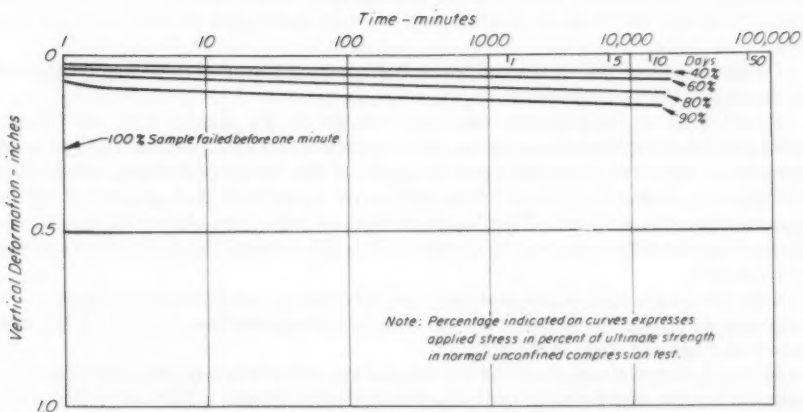


FIG. 27.—DEFORMATION VS TIME CURVES IN SUSTAINED LOAD CREEP STRENGTH TESTS ON WYOMING BENTONITE.

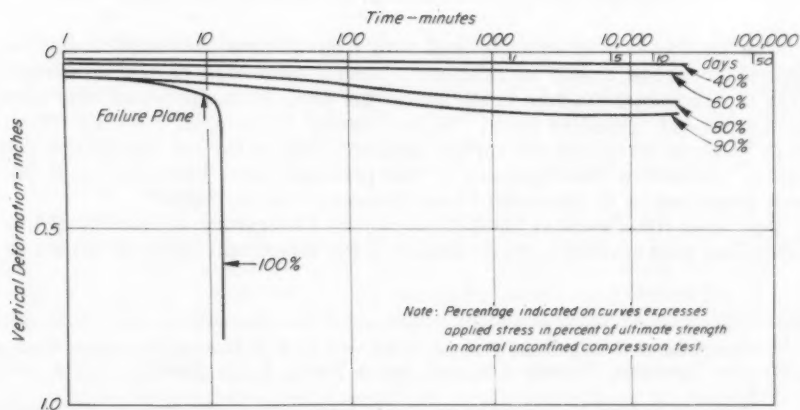


FIG. 28.—DEFORMATION VS TIME CURVES IN REPEATED LOAD CREEP STRENGTH TESTS ON WYOMING BENTONITE.

at the higher load values, whereas at the lower loads the tendency is for the sustained load to produce more creep deformation than the equivalent repeated load.

In both cases, the rate of creep shows a tendency to increase with the increasing load; at least until a more stable matrix structure is built up. The question of the build-up of matrix structure is dealt with in the following section.

On the basis of the above evidence the bentonite may, therefore, be classed as repeated-load-insensitive.

It follows that, in general, for a soil in which the matrix has not yielded, sustained loads will produce more deformation than repeated loads of the same magnitude, whereas once the yield strength of the matrix has been reached the deformation under a repeated load will tend to exceed that produced by the equivalent sustained load. This behavior has, in fact, been observed in previous studies on the silty clay and is shown in Fig. 29 (unpublished information from H. B. Seed).

It has been found that the matrix yield strength as determined from this diagram shows good agreement with the values estimated from studies of the type shown in Figs. 13 and 19 through 21.

At the present time, there is an increasing awareness of the importance of considering the behavior of soils under repeated loads. This is particularly true in the realm of pavement design. Observations such as those reported by Seed, et al.,<sup>13</sup> and F. N. Hveem,<sup>14</sup> have shown the significant differences between sustained and repeated load behavior. It is, thus, likely that the problem of assessing the sensitivity of a soil to the application of repeated loads will assume increasing importance, particularly when "failure" is determined as the stress required to produce a maximum allowable strain. Recognition of the step-strain phenomenon and strength studies, such as those described in this paper, may therefore be of some value in helping to provide a better solution to some road-building problems.

### THIXOTROPY AND STRENGTH GAIN UNDER LOAD

The phenomenon of strength gain with time of remolded soils under zero applied stress has, in soil mechanics literature, come to be called "thixotropy." This is in contradistinction to the use of the term in chemistry circles to describe sol-gel transformation. In the present discussion, the word "thixotropy" will be taken with the former meaning, that is, the soil mechanics definition. Extensive investigations of this phenomenon in saturated clays have been presented by O. Morretto,<sup>15</sup> and Skempton and Northey.<sup>8</sup>

For some time, work at the Soil Mechanics Laboratory, University of California, has been devoted towards studies of the thixotropic behavior of partially

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<sup>14</sup> "Pavement Deflections and Fatigue Failures," by F. N. Hveem, Design and Testing of Flexible Pavement, Highway Research Board, Wash., D. C., Bulletin no. 114, 1955, pp. 43-72.

<sup>15</sup> "Effect of Natural Hardening on the Unconfined Compression Strength of Remolded Clays," by O. Morretts, Proceedings, Second Internatl. Conf. on Soil Mechanics and Foundation Engrg., Vol. 1, 1948, pp. 137-144.

saturated soils and the effects of repeated and sustained loading on the subsequent deformation characteristics of compacted clays. Seed and C. K. Chan<sup>16</sup> have shown that appreciable strength increases may occur in partially saturated soils after a period of rest, and considerable increases in stiffness and strength of a compacted clay have been shown to result from a series of applications of a relatively light stress.

From the general theory of soil structure developed in this paper, an explanation of the thixotropic phenomenon can be put forward, and the associated behavior in which a compacted soil gains strength under the application of loads which are insufficient to cause failure can also be explained in similar terms.

According to the hypothesis of soil structure presented in the first part of this paper, any variation in the strength of the colloidal matrix in a constant stress environment can only be accomplished if the mean effective distance between particles is changed. Thus, it is suggested that the thixotropic strength gain is due to changes in disposition of the colloidal particles within the soil mass.

Clearly, if this suggestion is to be tenable, the proposed changes must be accomplished through the redistribution of internal (that is, inter-particle) stresses.

In the preparation of a compacted soil at a given water content, it is most unlikely that the most stable colloidal structure will be immediately established everywhere. Every soil sample which is not in intimate contact with a supply of free water at atmospheric pressure must be subject to a moisture deficit which manifests itself as an apparent tension in the pore water. It is again most unlikely that in the preparation of such a sample an equilibrium distribution of moisture tension will be immediately attained.

For soils prepared at a high degree of saturation (with domestic water), the associated moisture tension is of low magnitude, and, thus, the forces which determine the particle arrangements are the inter-particle repulsive forces. In these circumstances the particles are under insignificant external restraint and are free to alter their dispositions to achieve maximum stability within the given environment. Consequently, the available shear strength increases with time as the most stable colloidal structure is built up.

When, however, the soil is prepared at a low degree of saturation, the particles are continually within an environment where the net inter-particle force is attractive. Hence, a structure, of the type shown in Fig. 7, is established during the preparation. Thereafter, there is no mechanism whereby this structure can be varied without altering the external stress conditions. (This conclusion is in line with that developed by Lambe<sup>3</sup> in a recent publication on the structure of compacted soils.)

This is put forward as an explanation of the fact that, as will be seen from Fig. 31, no thixotropic strength gain was observed in the bentonite samples under test. Skempton and Northey<sup>8</sup> have suggested that all soils lose this potential strength gain with time as the moisture content approaches the plastic limit of the soil concerned. The present hypothesis suggests that this is due

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<sup>16</sup> "Thixotropic Characteristics of Compacted Clays," by H. B. Seed and C. K. Chan, *Journal of the Soil Mechanics and Foundations Div., Proceedings of the American Society of Civil Engineers*, Vol. 83, No. SM 4, November, 1957, pp. 1-35.

to the progressive increase of moisture tension as the moisture content decreases to the critical value below which no significant particle reorientation can occur, and that the plastic limit is, per se, not related to this behavior.

It should be emphasized that the behavior discussed previously is assumed to occur at constant volume of the sample as a whole and is not to be confused with the gain in strength with time due to pore pressure dissipation.

Seed, et al.,<sup>17</sup> have observed that significant increases of strength are developed in a compacted soil at constant water content and constant volume fol-

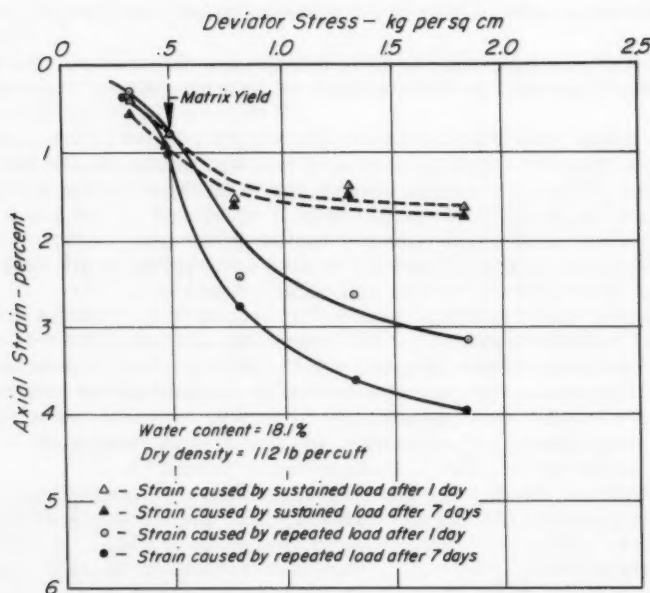


FIG. 29.—EFFECT OF MATRIX YIELD ON CREEP STRAIN UNDER SUSTAINED AND REPEATED LOADS.

lowing the application of subfailure loads for considerable periods of time. They have offered convincing evidence that this gain in strength cannot be accounted for in terms of density changes and/or over-all moisture variation and have inferred that the effect is due to the creation of some type of structure in the soil.

<sup>17</sup> "Increased Resistance to Deformation of Clay Caused by Repeated Loading," by H. B. Seed, R. McNeill and J. DeGuenin, Journal of the Soil Mechanics and Foundations Div., ASCE, Vol. 84, No. SM 2, May, 1958, pp. 1-35.

Further evidence of this increase in strength is presented in Figs. 30 and 31, which show the results of the normal unconfined compressive strength tests carried out on the silty clay and bentonite after the samples concerned had been subjected to the various creep-strength tests. In Figs. 32 and 33, an attempt to plot the general trends of this strength increase is shown. To date this

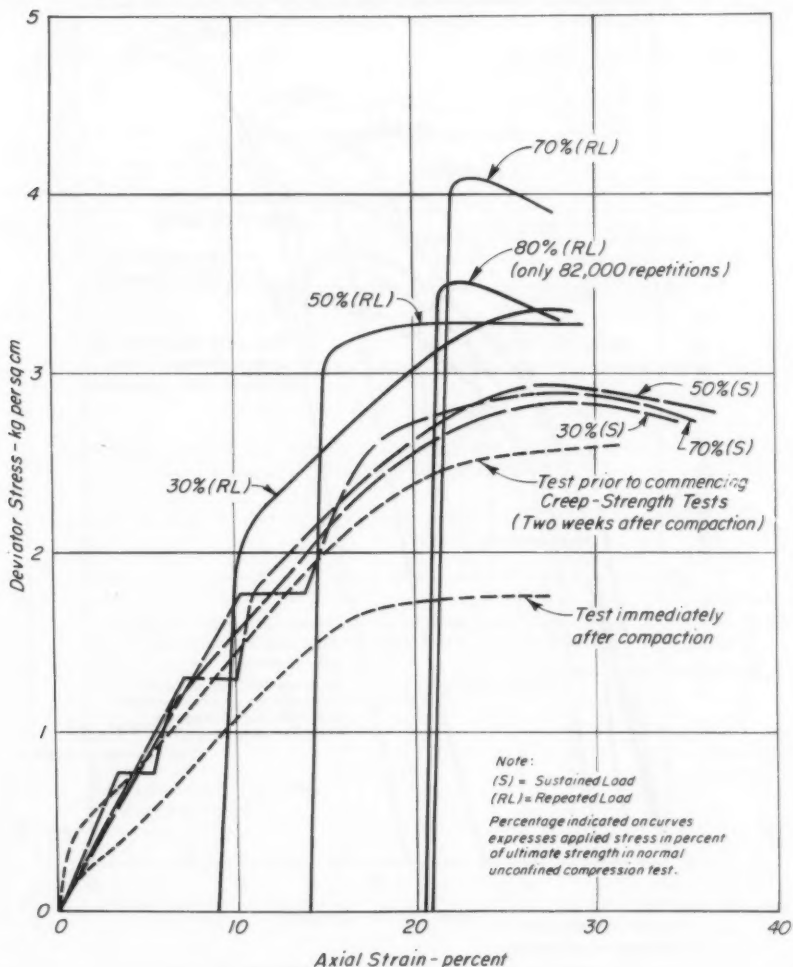


FIG. 30.—STRESS VS STRAIN RELATIONSHIPS OF SIMILAR SAMPLES OF SILTY CLAY AFTER BEING SUBJECTED TO CREEP-STRENGTH TESTS FOR 10 DAYS.

method of plotting—wherein the abscissas are the product of the stress applied in the creep-strength test and the logarithm of its time of application—is the only one which has been found to show any consistent trend in terms of applied stresses. Despite the obvious complexity of this behavior, one significant factor does emerge from this study. The silty clay shows a considerably different gain of strength under repeated load application than under sustained loads,

whereas the bentonite does not show such a difference except where the loads applied in the creep strength test are relatively high. It may be inferred, therefore, that the structure of the silty clay differs from that of the bentonite.

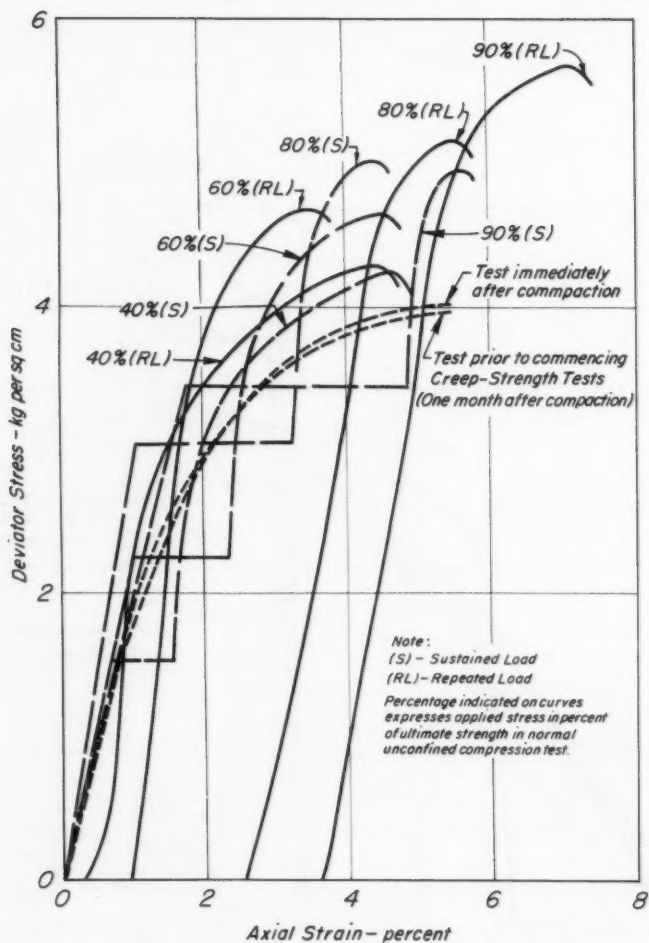


FIG. 31.—STRESS VS STRAIN RELATIONSHIPS OF SIMILAR SAMPLES OF WYOMING BENTONITE AFTER BEING SUBJECTED TO CREEP-STRENGTH TESTS FOR 10 DAYS.

Thus, evidence gives added support to the conclusions reached earlier in this paper—that the silty clay is repeated-load sensitive, whereas the bentonite is not. Within the framework of present knowledge of soil structure, the gain in

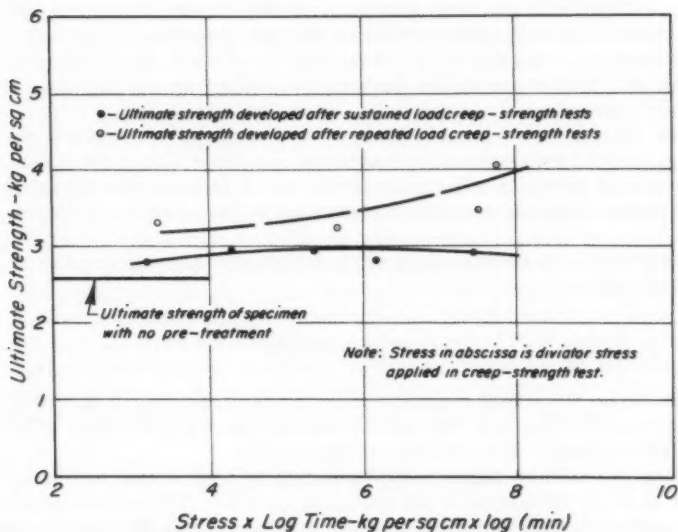


FIG. 32.—EFFECT OF LOAD AND TIME ON STRENGTH GAIN OF SILTY CLAY IN CREEP-STRENGTH TESTS.

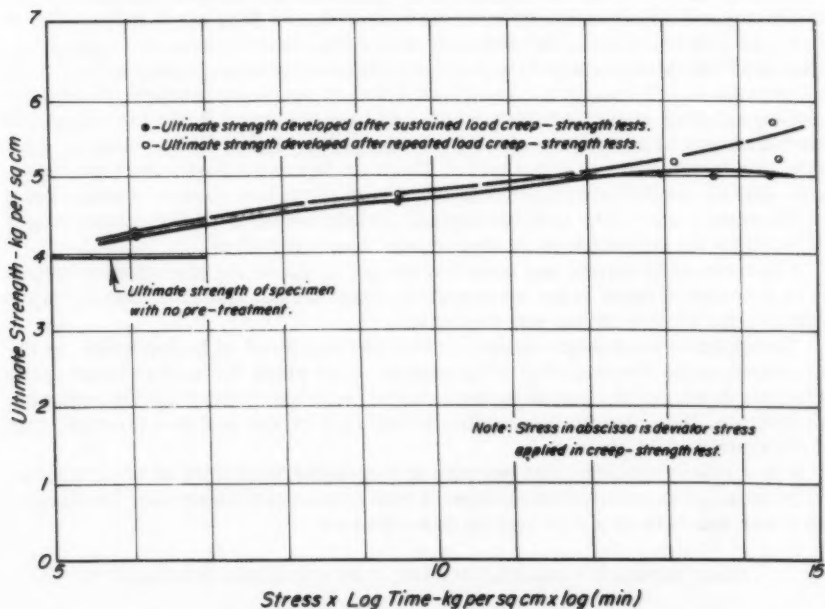


FIG. 33.—EFFECT OF LOAD AND TIME ON STRENGTH GAIN OF WYOMING BENTONITE IN CREEP-STRENGTH TESTS.

strength can be accounted for in general terms of structure build up (decreasing the mean particle spacing) and the associated pore pressure dissipation. In a saturated soil it is clear that these factors are interdependent.

Furthermore, from the evidence presented by Seed, et al., it appears that these changes must occur within the potential failure zone rather than throughout the soil mass.

It may be concluded, therefore, that the results of these tests give added emphasis to the importance of soil structure in determining the shear resistance of soils at constant volume. In particular, it is seen that the development or breakdown of structure within the potential failure zone is of major significance.

These factors can be of considerable assistance in assessing the long term behavior of soils.

### CONCLUSIONS

On the basis of the soil structure hypothesis presented, it has been postulated that a composite soil can be represented as coarse (sand, silt) grains distributed at random in a colloidal (clay) matrix.

When such a soil is subjected to sufficiently high shear stresses a yield condition is first developed in the colloidal matrix. This condition of matrix yield is achieved when the applied shear stresses are sufficient to force the colloid particles into such a position that they tend to be oriented parallel to one another and in the direction of the potential failure surface. Subsequently, it is thought that the coarser grains migrate in such a way that additional shear resistance, in the form of intergranular friction, is developed in the zone of potential failure. Ultimately, complete shear failure will occur when the maximum possible intergranular friction is developed in addition to the yield strength of the matrix. This mechanism is put forward as an explanation of the phenomenon of step-strain behavior which has been observed in the laboratory. In the latter part of the paper, some experimental evidence, which seeks to support this hypothesis, is submitted. Tests on Wyoming bentonite have shown what may be described as typical matrix characteristics whereas similar tests on Vicksburg silty clay indicate typical matrix behavior in the initial stages followed by the development of step-strain characteristics.

The term step-strain has been introduced because the phenomenon shows up as a series of steps in the stress-strain diagram and also as a step in deformation time studies of the soil concerned.

The appearance of step-strain behavior is considered to be dependent on the structural order (sensitivity) of the matrix. If, at yield, the matrix loses more strength, due to local remolding, than can be replaced by developed intergranular friction, then clearly the maximum strength of the soil is a function only of the matrix yield strength.

It is further suggested that because of a possible similarity of basic mechanisms entailed in relative translation of both coarse and fine grained particles, the shear strength of a soil may be described as:

$$\text{shear strength} = \text{colloidal friction} + \text{intergranular friction}$$

The study of these aspects of soil structure led to the conclusion that the nature of the matrix would have a considerable influence on the creep behavior of a soil and also on its response to repeated loading.

Tests have tended to show that whereas a soil which exhibits step-strain characteristics is liable to develop lower strengths under long term loading, then in normal unconfined compression tests (creep-strength loss), a soil that develops its strength only from the matrix does not show a similar loss.

Furthermore, soils exhibiting step-strain characteristics also show much greater deformation under repeated loads. Thus, recognition of this phenomenon may facilitate identification of repeated-load sensitive soils.

The importance of rate and sequence of loading on the ultimate strength measured under conditions of no over-all volume change has also been demonstrated. Tests have tended to show that if subfailure loads are applied to a soil specimen, under these loads the shear resistance of the soil increases with time. This has been attributed to improvement in the colloidal structure and to the associated dissipation of pore pressures in the potential failure zone.

It is clear that many factors influence the strength and deformation characteristics of soils when stressed at constant volume. The conclusion of greatest practical significance, however, is that the variation in ultimate strength of a given clay-type soil, under differing stress conditions, is largely determined by the characteristics of the colloid fraction of the soil.

#### ACKNOWLEDGMENTS

In the spring of 1957, the first writer had the privilege of working as a visitor on sabbatical leave in the Soil Mechanics Laboratory of the University of California, Berkeley. It was during this period that the phenomenon described herein as the step-strain phenomenon was observed and studied.

The experimental work described in this paper was carried out at the Soil Mechanics Laboratory of the University of California, Berkeley.

The writers particularly acknowledge the cooperation of H. B. Seed for making laboratory facilities and personal records of previous work available, as well as for reading the manuscript and making many helpful suggestions. L. Shifley and C. Swartz assisted in carrying out the tests and G. Dierking prepared the diagrams.



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Journal of the  
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Proceedings of the American Society of Civil Engineers

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PILE DRIVING EXPERIENCES AT PORT EVERGLADES

By T. J. Lynch,<sup>1</sup> M. ASCE

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SYNOPSIS

Material settlements of piles after driving to practical refusal but before placing loads thereon was an unusual circumstance which developed during the construction of foundations for the Port Everglades Plant. This paper outlines the soil investigations, the difficulties encountered in the driving of piles, and the pile settlements. The conclusion is reached that the settlements resulted from consolidation of underlying strata attributable to vibrations developed by pile driving.

Pile-load test data are presented and records of comparative driving with an air hammer and a rated equivalent (30,000 lb-ft) diesel hammer are included.

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SCOPE OF THE PROJECT

The Port Everglades Power Plant consists of two 240,000 kw steam generating units complete with all necessary auxiliaries. The site selected for this plant comprises an 80-acre plot located about  $\frac{1}{2}$  mile south of the developed area of Port Everglades, Fla. Fig. 1 shows the site location. Approximately one-half of the site was entirely undeveloped and consisted of a dense mangrove jungle swamp with ground surface in the tidal range at El. +2. The remainder of the site had been filled with imported lime sand to an approximate El. +7.

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Note.—Discussion open until September 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. SM 2, April, 1960.

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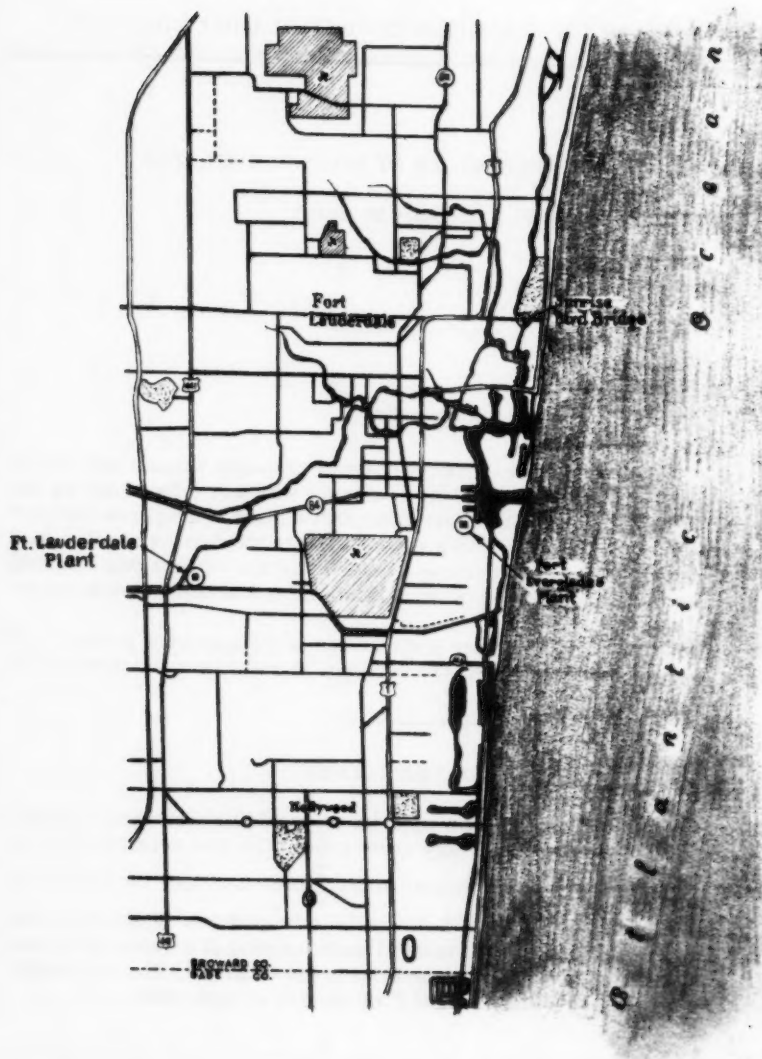


FIG. 1.—AREA MAP

Development of a general arrangement of the plant facilities on the site showed the most advantageous location for the main plant structures to be in the initially undeveloped portion with the previously filled area reserved for coal storage.

To protect the plant against potential hurricane floods, it was decided that the ground floor level for the plant be El. +20. This necessitated importation of approximately 350,000 cu yd of fill material for the initial two unit plant.

Major foundation loads for each unit are:

1. Turbine generator pedestal. A massive concrete structure; in plan about 40 ft by 100 ft; 24 ft high above finish grade; gross dead weight approximately 4,000 tons.
2. Boiler structure. A structural steel braced frame 80 ft by 100 ft in plan and 150 ft high; eight major columns with a maximum column load of 1,200 tons.
3. A reinforced concrete chimney 358 ft tall with an inside diameter of 14 ft at the top and designed for hurricane wind loading.

In addition to these, many auxiliary structures produce lighter concentrations of load and the weight of the imported fill material constitutes an appreciable surcharge.

#### INITIAL INVESTIGATIONS

*Geology.*—The general soil profile in Southern Florida consists of a surface layer of peat varying from a few inches to several feet in thickness and underlain by a limestone formation interbedded with layers of sand.<sup>2,3</sup> This limestone formation, known as Miami oolite, extends to an approximate elevation of -45 ft, is highly permeable and contains solution channels and layers of redeposited material.

Underlying the Miami oolite is the Fort Thompson formation, composed of marine and fresh water limestones and some quartz sand and shells. Together with the similarly composed Tamiami formation, this material extends downward to a depth of over 200 ft. These limestone formations are unconsolidated alluvial material and may be of low density, particularly in the surface strata. However, where redeposited material (calcite) acts as a cementing agent, layers of considerable density occur.

*Neighboring Structures.*—In the areas immediately adjacent to the site of the Port Everglades Plant, there exist a number of oil storage tanks and a large multiple cement silo which impose appreciably foundation loads. Oil storage tanks are typically constructed on a sand fill placed after removing the surface peat. Tanks constructed in this manner settled slightly ( $1\frac{1}{2}$  in.) when first filled and have shown no further settlement.

The cement silos are constructed on a concrete mat resting on a compacted sand fill which was placed in the area of the foundation to a depth of 20 ft. This structure settled  $1\frac{1}{2}$  in. during construction and about 4 in. during the first 6 months after construction. No further settlement occurred in the following year.

<sup>2</sup> "Dewatering Miami's Biscayne Aquifer," by Byron J. Prugh, *Journal of Soil Mechanics, ASCE*, Vol. 83, July, 1957.

<sup>3</sup> "Unusual Foundation Conditions in the Everglades," by Paul H. Shea, *Transactions, ASCE*, Vol. 120, 1955, p. 101.

*Initial Borings, Soundings and Test Piles.*—At the time of signing the contract for the Port Everglades Plant, the contractor was completing the construction of Units 4 and 5 at the Fort Lauderdale Plant, located about six miles from the Port Everglades site (Fig. 1). At the Fort Lauderdale Plant, Units 1, 2 and 3, constructed in 1928, were founded on a mat on the Miami oolite at El. -5.0 and no material settlements had occurred. However, for Units 4 and 5, it was found more economical to drive piles with their tips extending slightly into the Fort Thompson formation, which is locally termed "rock."

The piles for the Fort Lauderdale Plant were 14-in. diameter Hel-Cor shells filled with concrete. Piles were driven without difficulty with an internal mechanically-expanding mandrel and a No. O Vulcan hammer and reached practical refusal at the top of the Fort Thompson formation.

As a result of this experience and with the information that the geology of the area was reasonably uniform, the preliminary soils investigation for the Port Everglades Plant was based on the assumption that the primary requirement was to determine the top of "rock," (Fort Thompson formation) and that such determination could be made by finding the point of refusal of the casing and/or sampling spoon.

To determine conditions at the site and thereby establish criteria for foundation design, borings and soundings were made during the fall of 1957. Borings were specified to be carried to, and extend, a minimum of 5 ft into rock. Soundings by penetrometer supplemented the borings in indicating the surface of the rock.

The borings and soundings indicated that the depth of acceptable bearing strata varied appreciably over the site, but would generally be encountered between El. -40 and El. -50. Below El. -25, soft and hard strata are interlain and irregular. Indication was that piles driven to a blow count equivalent to a 50-ton pile might take up in a hard layer underlain by appreciable thickness of compressible material.

Six test piles were driven at the more heavily loaded locations distributed in the area in which most piles would be located. To correlate driving results with the borings, additional borings were made at the test pile locations. The results were similar to those previously obtained. Three indicated "rock" at El. -40, two at El. -50, and one penetrated to El. -80, indicating dense sand at that level, but no "rock." The test piles all drove to tip elevations between El. -75 and El. -80. Only one of the six encountered appreciable resistance at the "rock" level, and that for only 2 ft. Resistance corresponding to that specified in the subsequently-issued specification was encountered by all six piles at levels between El. -68 and El. -75. Driving logs of these test piles are shown in Fig. 2.

These results, together with an analysis of the boring logs, indicated the desirability of extending the pile tips to an average of El. -70. To secure this result, the required "set" per blow by the Engineering News formula was reduced (on the assumption of a 50-ton bearing and a No. O Vulcan hammer) from 0.387 to 0.192 in. per blow or increasing the blows per foot of final penetration from 31 to 62.

Even with this modification, the possibility of reaching this resistance at a higher elevation was recognized. Therefore, a "Guide for Pile Inspector," stated,

"For the subsurface conditions at Port Everglades, it is especially important to have the 50-ton piles penetrate well into the dense layer occurring below Elev. (-)50 over most of the site. If hard driving is met at

a shallower elevation, it may be necessary to resort to jetting to get past the obstruction and into the lower dense layer. It is expected that most of the 50-ton piles will be driven down between Elev. -60 and -70."

*Initial Conclusions.*—From the subsurface investigation, conclusions made prior to start of work on the site were as follows:

1) Due to the compressible nature of the strata immediately below the surface, piles are necessary to prevent large settlements.

2) Timber piles are uneconomical due to limited capacity. Steel H-piles are unsuitable due to corrosion danger at the ground water surface. Concrete piles, either cast in place or precast, are acceptable as well as being most economical.

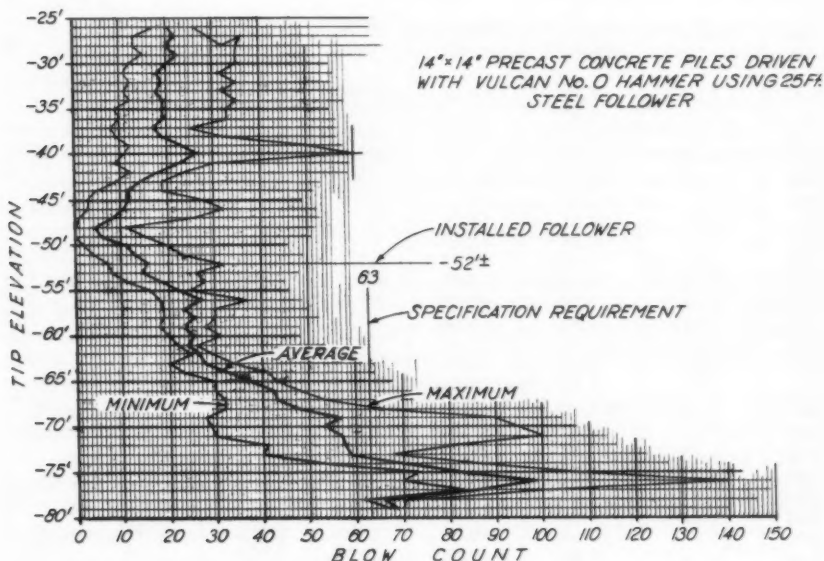


FIG. 2.—TEST PILE DRIVING LOGS

3) Piles should penetrate to the dense material below El. -60. No unusual difficulty should be encountered in achieving this penetration.

4) No additional investigation is warranted.

### DESIGN

The design of foundations for the plant progressed on the basis of a 50-ton per pile design capacity. Of this, 10 tons was considered as an allowance for negative skin friction in the fill and upper sand strata. Since the fill was placed with no compaction directly over the unconsolidated muck layer, it was considered necessary to support even the lightest structures on piles.

## CONSTRUCTION

*Piling Specification and Supplementary Instructions.*—On June 12, 1958, a subcontract was awarded, based on cast-in-place concrete piles using Hel-Cor shells. Pile driving commenced on July 7, 1958, utilizing an internal, mechanically expanding mandrel and a Vulcan No. O hammer.

From the outset, attempts at driving the shells for piles encountered difficulties. In many cases, the shells were damaged by driving and the specified blow count was reached prior to obtaining the desired penetration. Although occasional piles could be allowed to stop short, it soon became apparent that

TABLE 1<sup>a</sup>

Pile No.	Initial 9-10-58	Final 11-6-58	Settle- ment	Pile No.	Initial 10-2-58	Final 11-6-58	Settle- ment
W7	7.37	7.23	-0.14	W103	7.771	7.755	-0.02
W9	7.37	7.24	-0.13	W105	7.646	7.570	-0.08
W10	7.37	7.38	+0.01	W106	7.429	7.365	-0.06
W12	7.37	7.20	-0.17	W108	7.468	7.480	+0.01
W13	7.37	7.38	+0.01	W109	7.415	7.352	-0.06
W15	7.37	7.27	-0.10	W111	7.418	7.419	0
W16	7.37	7.41	+0.04	W112	7.114	7.048	-0.06
W18	7.37	7.29	-0.08	W114	7.068	7.068	0
W19	7.37	7.40	+0.03	W115	6.975	7.885	-0.09
W21	7.37	7.34	-0.03	W117	6.977	6.995	+0.02
W22	7.37	7.44	+0.07	W118	6.785	6.755	-0.03
W24	7.37	7.03	-0.34 <sup>b</sup>	W120	6.701	6.694	-0.01
W25	7.37	7.33	-0.04	W121	6.659	6.621	-0.04
W27	7.37	7.29	-0.08	W123	6.606	6.608	0
W28	7.37	7.28	-0.09	W124	6.363	6.361	0
W30	7.37	7.27	-0.10	W126	6.316	6.311	-0.01
W31	7.37	7.39	+0.02	W127	6.248	6.239	-0.01
W33	7.37	7.12	-0.25 <sup>b</sup>	W129	6.219	6.222	0
W34	7.37	7.29	-0.08	W130	5.962	5.965	0
W36	7.37	7.21	-0.16	W132	5.988	5.983	-0.01
W37	7.37	7.32	-0.05	W133	5.957	5.950	-0.01
W39	7.37	7.29	-0.08	W135	5.851	5.841	-0.01
W40	7.34	7.30	-0.04	W136	5.554	5.592	+0.04
W42	7.34	7.25	-0.09	W138	5.692	5.655	-0.02
W43	7.25	7.04	-0.21	W139	5.489	5.500	+0.01
W45	7.25	7.25	-0.00	W141	5.529	5.544	+0.02
W46	7.18	6.94	-0.24 <sup>b</sup>				
W48	7.18	6.99	-0.19				

<sup>a</sup> All values are in feet.    <sup>b</sup> Maximum values.

the trouble was widespread. Initial evaluation of the problem was clouded by difficulties with adjustment and operation of the driving equipment. These conditions were rectified by early August.

It then became obvious, at least in some areas, that driving conditions were substantially more difficult than had been anticipated. Experimentation was conducted in the field aimed at devising a satisfactory and economical method of placing the piles. A heavier hammer, Vulcan OR, and heavier gage material for the shells (14 and 16 rather than 18 gage) were adopted to secure better penetration. Intermittent welds were added to the shell crimped seams, found

to be quite beneficial and adopted for the remainder of the job. Jetting and drilling, to obtain penetration through the hard layers, were attempted and, although found to be of some value, were not adopted because of excessive cost, complication and operational difficulties.

Driving of pipe piles (12-in. diameter, 3/8 in. wall) was attempted in the hard areas and found to be quite successful. Based on this, it was decided that pipe piles would be used in areas where satisfactory penetration could not be achieved with the shell type piles.

*Pile Settlement.*—Prior to September 21, 1958, some pile caps to support 66-in. diameter, concrete-circulating water pipes were poured and a portion of the pipe was put in place. During subsequent driving in the area adjacent to these caps, settlements of the caps were observed. Details of these settlements are included in Tables 1 and 2. The maximum settlement noted was approximately 7 in.

To discover the reasons for these settlements, it was decided that additional information was required. To aid in this investigation, a foundation consultant was retained.

*Subsequent Borings.*—After the observed settlement of previously-driven piles supporting the circulating water pipes, it was evidently necessary to determine the reason for the settlements and to establish that such settlements were not a result of unsatisfactory subsoil conditions at a depth greater than previously explored. Accordingly, the pile load test, described in the following paragraphs, and the borings, the results of which are shown on Fig. 3, were made.

This subsequent boring program generally confirmed the findings of the previous exploration and the above conclusions that the piles should extend to an average of El. -70.0. It developed the further information that all strata below this elevation were so compact that no appreciable settlements would occur as a result of their consolidation under load.

*Pile Load Tests.*—Two piles, C 13 and C 16, both having 14-in. Hel-Cor shells, were driven to tip elevations of El. -66.0 and El. -67.0, respectively. The location of these and the surrounding piles in the area is shown in Fig. 4. In each of these two piles (Fig. 5), 1-in. diameter pipes were inserted with tips 28 ft, 56 ft, and 78 ft from the top into which rods could be dropped. This arrangement is shown on Fig. 5.

The purposes of these tests were to:

- 1) Remove any question as to the ultimate capacity of the piles.
- 2) Investigate the effects of driving adjacent piles on a loaded pile and to discover the distance within which such effects occurred.
- 3) Investigate the amount of load taken by skin friction at the various elevations and the amount taken by end bearing.

The piles were loaded by jacks as shown by Fig. 6. The arrangement was such that a load of 100 tons could be applied to each of the two piles simultaneously; or with one jack not being used, 150 tons could be applied to either of the two piles. Jacking was against a known weight.

The pile driving records for the two piles and a soil profile at their location (the latter being interpolated from adjacent borings) are shown on Fig. 7. The data from the loading tests are shown in Table 3.

Table 4 gives additional data as to the settlements produced by driving adjacent piles and also an indication from the 3 ft rods of the horizontal distance over which this effect occurs.

TABLE 2a

Pile No.	10-1-58	10-15-58	11-1-58	11-7-58	11-19-58	12-1-58	12-12-58	Total Settle- ment
H5	12.22	12.213	12.167	12.153	12.153	-	-	0.07
H8	-	12.18	12.146	12.137	12.140	-	-	-0.04
H17	12.45	11.899 <sup>b</sup>	11.802	11.653	12.465 <sup>b</sup>	-	-	-0.58 <sup>c</sup>
H20	-	12.444	12.232	11.887 <sup>b</sup>	12.409 <sup>b</sup>	-	-	-0.21
H29R	-	12.099	12.055	11.761 <sup>b</sup>	-	-	-	-0.04
H32	12.355	12.345	12.331	12.321	-	-	-	-0.03
H41	-	12.111	12.075	-	-	-	-	-0.03
H44	12.314	12.282	12.262	12.253	-	-	-	-0.06
H50	12.241	12.208	12.183	12.160	-	-	-	-0.08
H56	12.260	12.190	12.177	12.165	-	-	-	-0.10
H65	12.319	12.316	12.295	12.288	-	-	-	-0.03
H67	12.370	12.362	12.348	12.340	-	-	-	-0.03
H81	-	-	12.518	12.505	-	12.493	12.489	-0.03
H84	-	12.598	12.585	12.550	-	12.572	-	-0.03
H93	-	-	12.405	12.394	-	12.374	12.375	-0.03
H96	-	12.595	12.587	12.563	-	12.557	12.559	-0.04

a All values in feet. b Re-marked. c Maximum.

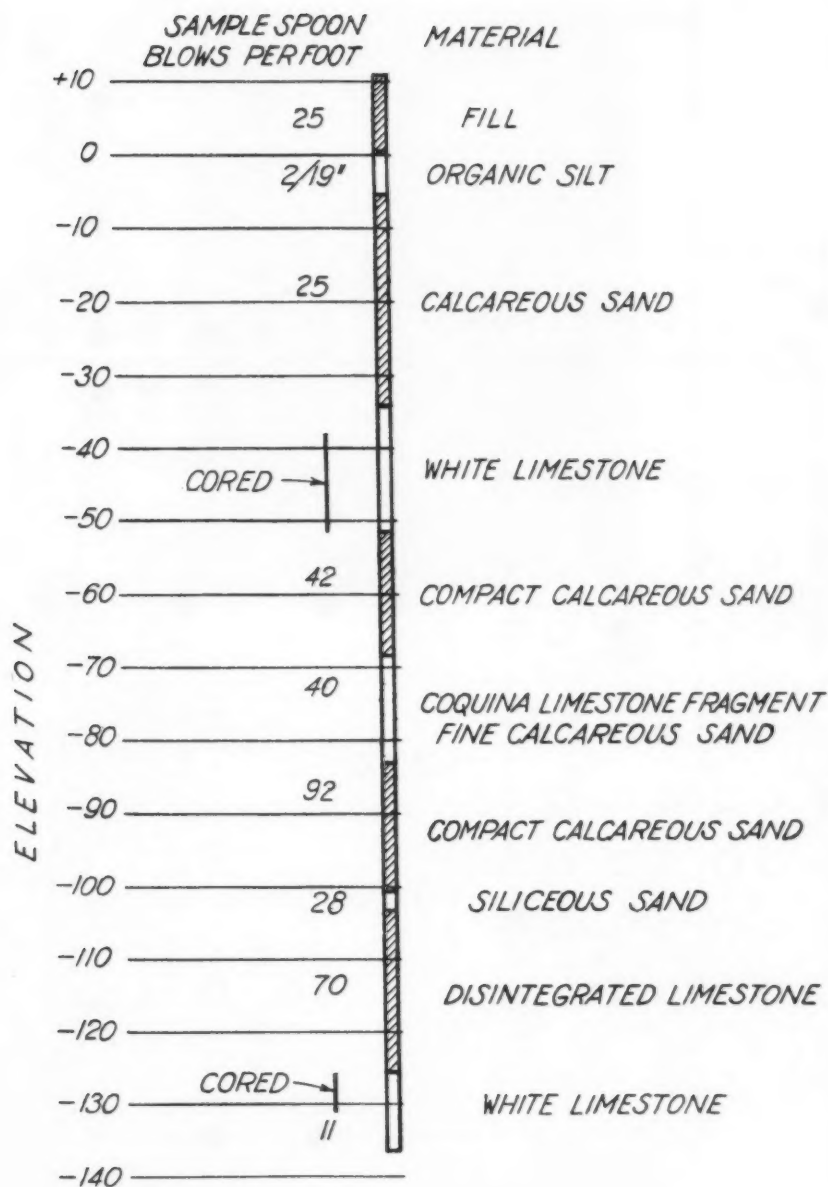


FIG. 3.—TYPICAL SUPPLEMENTAL BORING LOG

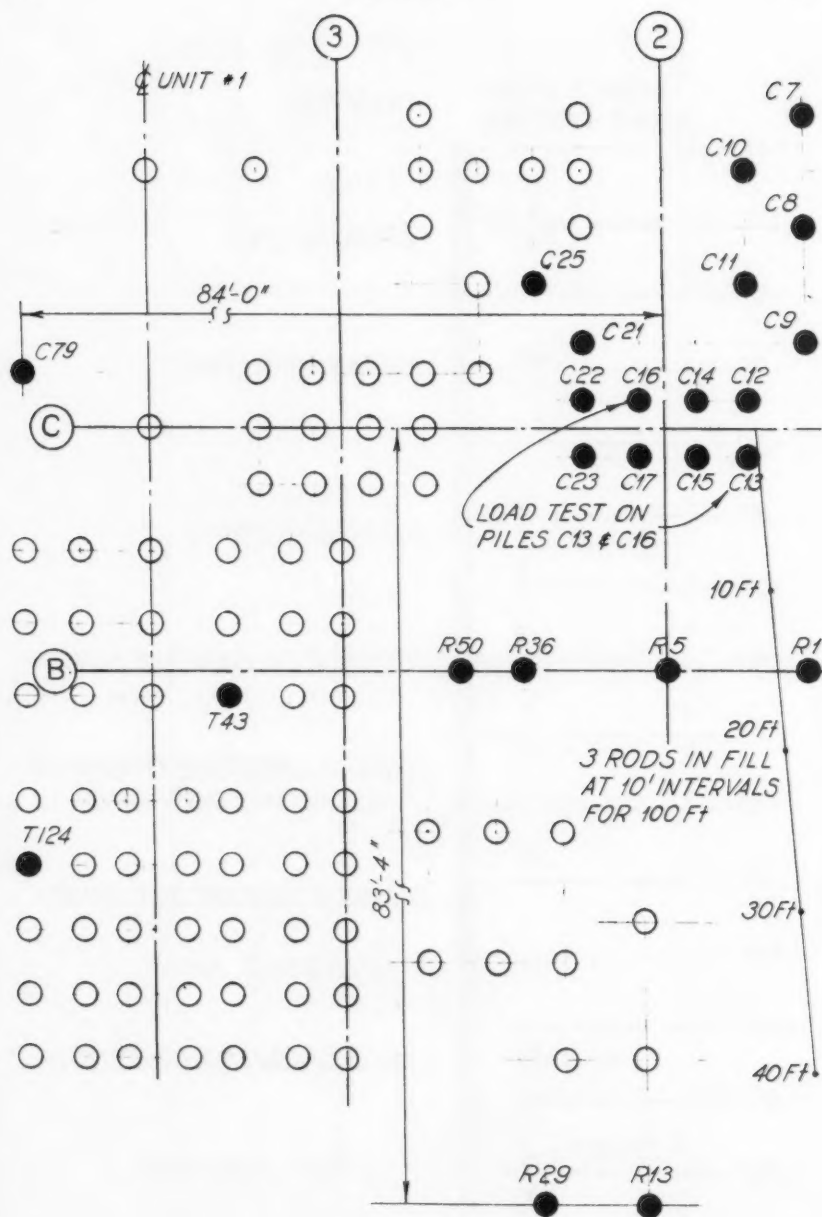
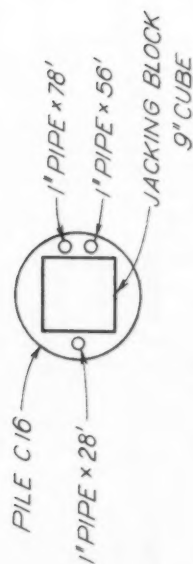


FIG. 4.—PILE LOCATIONS



OPR HAND FOR PILE C13

PLAN

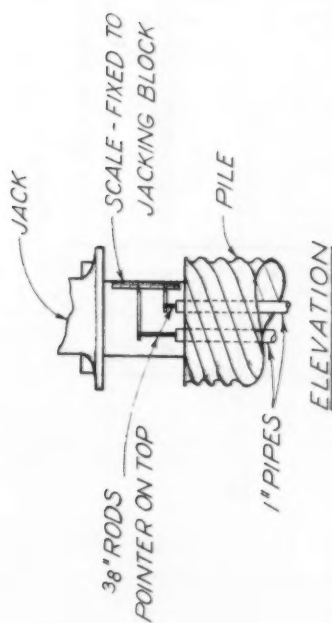


FIG. 5.—DEFLECTION MEASUREMENT DETAILS

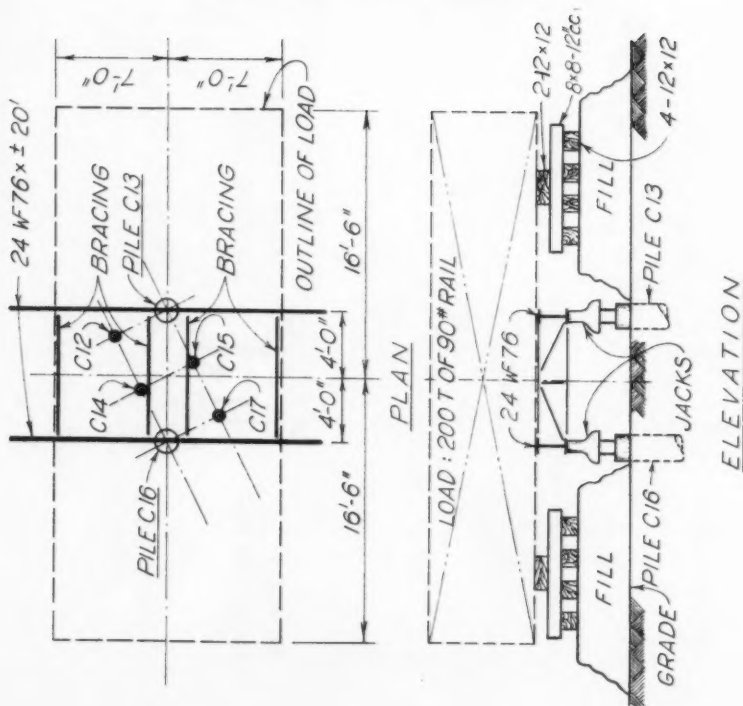


FIG. 6.—LOAD ARRANGEMENT

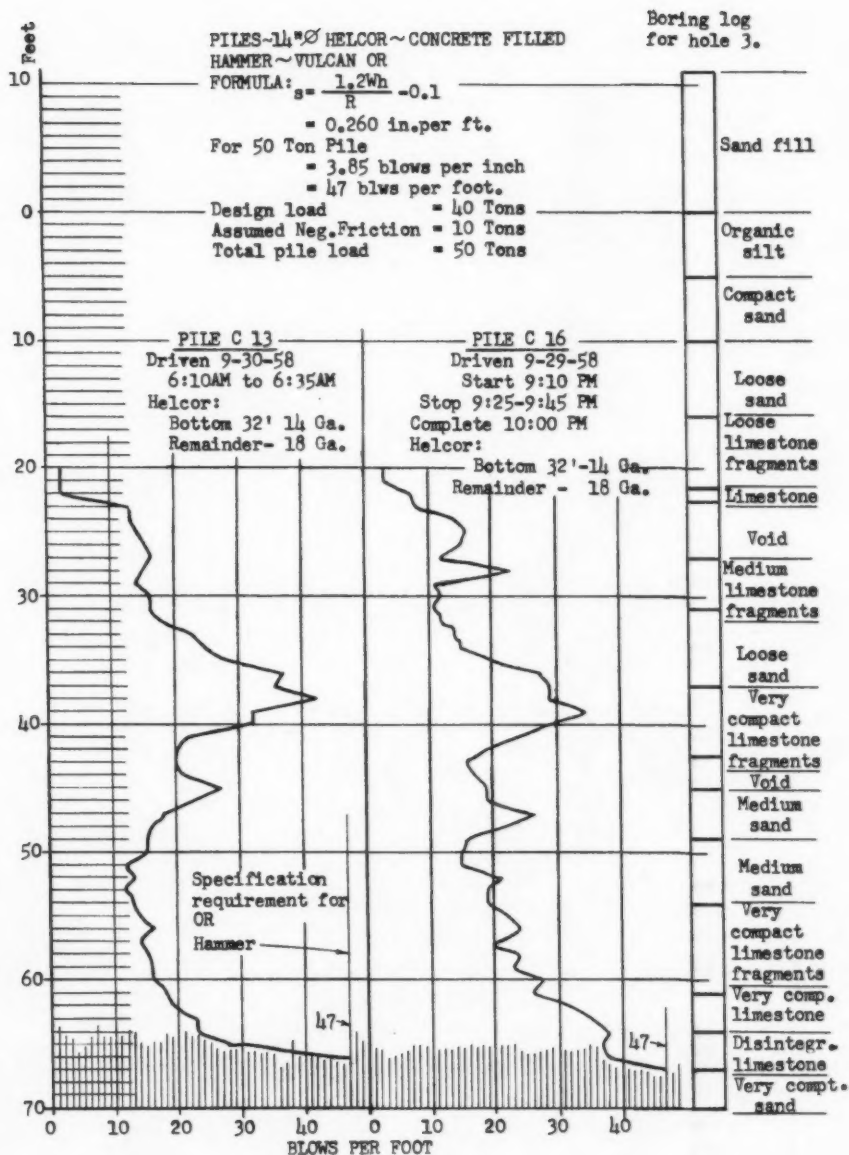


FIG. 7.—DRIVING LOGS - LOAD TEST PILES

TABLE 3.-PILE LOAD TEST DATA - PILES C13 &amp; C16a

C13				C16				
Top	78 ft	56 ft	28 ft		Top	78 ft	56 ft	28 ft
12' -7 11/32"	0	0	0	Driving Piles Around 10-6 Initial Reading " After Driving R36 " After Driving R50 10-7 After Driving C22 10-8 Standing Overnite	12' -7 9/32"	0	0	0
-12	-9	-10	-10		-18	-13	-15	-17
-18	-11	-14	-16		-21	-13	-17	-19
-19	-8	-15	-15		-16	-12	-15	-15
-25	-17	-21	-23		-21	-15	-19	-20
12' -6 17/32"	0	0	0	Trial Load 55 Ton	12' -6 22/32"	0	0	0
0	+2	+2	0		-5	-3	-3	-5
0	0	0	0		0	0	0	0
12' -6 19/32"	0	0	0	Piles Loaded as Indicated	12' -6 20/32"	0	0	0
-5	0	-1	-3		-3	-1	-1	0
-1	+1	0	0		-1	-1	-1	-1
-7	-1	-1	-5		-6	-1	-2	-2
-2	0	-1	-2		-1	-1	+3	0
-10	-1	-2	-8		-7	-1	-3	0
-2	0	0	-2		-1	-1	-1	0
-12	-1	-2	-8		-10	-2	-4	-3
-5	-1	-2	-5		-3	-2	-2	0
-5	-1	-2	-		-16	-2	-6	-6
-4	0	-1	-3		-3	-1	-2	0
-13	-1	-2	-8	Driving Adjacent Piles	-12	-2	-4	-5
-14	-2	-4	-10		-13	-3	-5	-4
-15	-2	-4	-11		-13	-2	-5	-4
-16	-3	-5	-12		-15	-4	-7	-6
-16	-3	-5	-12		-15	-4	-7	-6
-21	-4	-8	-17		-19	-4	-9	-8

a All figures indicate 32nds of an inch and show displacement from initial position.

Piles C-13 and C-16 were driven on September 29 and 30. The load tests started October 8, the concrete being only 4 days old at the start of the tests and 9 days old on their completion. At such ages, it is not possible to estimate, with any accuracy, the modulus of elasticity of the concrete. The assumption of 2,000,000 psi has been used in analyzing the data. Under this assumption, a load of 100 tons on the 14-in. diameter pile would produce shortening of 0.00065 in. per in. With concrete of this age, considerable plastic flow under load may be anticipated. Shrinkage of the concrete also may be a factor.

#### Discussion of Data:

##### 1) Driving:

The pile-driving logs, Fig. 7, are typical of the average driving, except that in many cases the resistance at El. -40.0 was such as to give considerably

TABLE 4.—SETTLEMENT DATA - ADJACENT PILES<sup>a,b,c</sup>

Location	Initial	After Driving C9	After Driving C11	After Driving R21	After Driving R36	After Driving R1	After Driving R50	Total Difference
C9	—	12.499	12.481	12.476	12.462	12.442	12.437	0.062 <sup>d</sup>
C11	—	—	12.591	12.585	12.557	12.542	12.537	0.054 <sup>d</sup>
C12	12.469	12.462	—	—	12.427	12.417	12.412	0.057
C13	12.612	12.606	12.606	12.601	12.532	12.570	12.564	0.048
C14	12.357	—	—	—	12.312	—	12.382	0.075 <sup>d</sup>
C15	12.464	12.461	—	—	12.427	12.422	12.417	0.047
C16	12.606	—	12.603	12.596	12.560	12.557	12.552	0.054
C17	14.985	—	—	—	14.952	—	—	0.033
C21	—	—	—	12.401	12.383	12.350	12.342	0.059 <sup>d</sup>
C23	13.373	—	—	—	13.342	13.307	—	0.066 <sup>d</sup>
R5	—	—	—	—	13.612	—	—	—
10	12.269	3 ft rods driven into ground. Rods driven before driving pile C9.					12.222	0.047
20	11.894						11.877	0.017
30	12.196						12.187	0.009
40	12.119						12.117	0.002
50	9.132						9.130	0.002
80	9.704						9.702	0.002
100	12.069						12.067	0.002
T43	9.917						9.909	0.008
T124	5.305						5.309	+0.004
C79	5.961						5.963	+0.002 <sup>e</sup>
R13	7.238						7.241	+0.003
R29	7.352						7.356	+0.004

<sup>a</sup> Piles are concrete-filled except as noted. <sup>b</sup> Elevations indicated are readings taken on piles as indicated during driving of piles around the test load location but prior to placing the load. <sup>c</sup> Order of driving: (1) C9, (2) C11, (3) C21, (4) C25, (5) R5, (6) R36, (7) R1, (8) R50. <sup>d</sup> Empty shell. <sup>e</sup> Pipe.

more difficulty than in the case of these two piles. With these two piles, the specified resistance was not reached at that level as was the case for many other piles.

##### 2) Load Tests:

Displacement readings for various loading and driving conditions are shown in Table 3.

TABLE 5. - PILE DEFORMATIONS

Distance from Top	C13				C16				Driving 3 Adjacent Piles
	78 ft	56 ft	28 ft	0	78 ft	56 ft	28 ft	0	
Settlement, 32nds				25					
Δ " "	17	21	23	2	15	19	20	1	21
Δ " , Inches				0.062					
Pile Shortening, In./In.				0.000185					
Average Load, Tons	4	0.125	0.062	0.062	4	0.125	0.031	0.031	14
	0.00058	0.00058	0.000185	0.000185	0.00058	0.00058	0.00009	0.00009	0.00009
	89	89	28	28	89	89	14	14	14
Settlement, 32nds				12					
Δ " "	1	2	8	4	2	4	3	7	10
Δ " , Inches				0.125					
Shortening, Ins./In.				0.00037					
Average Load, Tons <sup>b</sup>	0.031	0.188	0.125	0.00056	a	-1	0.03	0.218	0.00068
	0.00015	0.00056	0.00037	57 <sup>c</sup>	0.00009	0.00009	0.00009	0.00068	105 <sup>c</sup>
	23	86	57 <sup>c</sup>		-14	-14			
Settlement, 32nds				-					
Δ " "	-	-	-	-	2	6	6	16	
Δ " , Inches					4			16	
Shortening, In./In.					0.125			0.312	
Average Load, Tons <sup>b</sup>					0.00058			0.00093	
					90			143	
Settlement, 32nds				8					
Δ " "	3	6	9	-1	2	5	3	4	7
Δ " , Inches				0.031					
Shortening, Ins./In.				0.00009					
Average Load, Tons <sup>b</sup>	0.094	0.094	0.000280	0.00009	0.094	0.062	0.000185	0.00037	
	0.000435	0.000280	0.00009	-14	0.000435	0.000185	0.00037	57	
	64	41	41		64	-28			
Rebound, 32nds				7					
Δ " "	0	0	3	4	0	2	3	7	
Δ " , Inches				0.125					
Elongation, In./In.				0.00037					
Average Release, Tons	0	0.094	0.000280	0.00037	0.062	0.031	0.00009	0.00037	
	0	0.000280	0.00037	57	0.00028	0.00009	0.00037	57	
	0	41	41		41	14			
Rebound, 32nds				-					
Δ " "	-	-	-	-	1	4	6	13	
Δ " , Inches					3				
Elongation, In./In.					0.094	0.062	0.000185	0.00068	
Average Release, Tons					0.00044	0.000185	0.00068	105	
					64	28			

a Evident error in measurement. b Additional load. c Average load near top of two piles.

The data from Table 3 is transferred to Table 5 in a form that shows the differential shortening in the lengths between the tips of the rod inserts. The settlements at the top and the tip of the two piles are plotted in Figs. 8 and 9.

(a) Driving adjacent piles—Piles C-13 and C-16 unloaded.

Before piles C-13 and C-16 were loaded, several piles were driven. Piles E-36, R-50 and C-22, being respectively 18 ft, 25 ft, and 3.5 ft from C-16 and 20 ft, 28 ft and 11 ft from C-13, were driven in that order, along with the others, to an average of El. -67.0. Although at a considerable distance away, the driving of the first of these had the greatest effect. The settlement at the tops of the piles was  $25/32$  in. and  $21/32$  in., respectively; that at the bottom  $17/32$  and  $15/32$  in. The shortening of the piles averaged  $7/32$  in., or 0.219 in., or 0.00023 in. per in. of length. With the assumed modulus of 2,000,000 psi, this corresponds to an average load of 35 tons per pile. From Table 5, it may be deduced that the intensity of load is greater near the tip of the pile.

The most plausible explanation of this is that of negative skin friction. The peat layer becomes consolidated from the vibration of the pile driving; the same is true of the unconsolidated sands below El. -40.0. As these layers consolidate, the fill and the hard stratum at El. -40.0 follow the resulting settlement and by skin friction transmit loads to the pile. The loads of 89 tons at the tips of the pile and 28 tons above El. -40.0 may be far from accurate but probably give a good qualitative picture.

It is interesting to note that a short time after settlements of previously driven piles attributable to pile driving were first observed at the Port Everglades site, work was started by the City of Fort Lauderdale on the replacement of a bridge at Sunrise Blvd. in that city (Fig. 1). This bridge crosses the Intracoastal canal at a point about 5 miles north of the plant site. Pile driving for construction of the new bridge caused appreciable and damaging settlements to one pier of the existing bridge. The closest of the new piles to this pier was 8 ft away.

(b) Additional loads of 100 tons per pile.

With the piles loaded as described previously from negative skin friction, an additional load of 100 tons was placed on each pile. The shortening of the two piles average  $11/32$  in., or 0.343 in. or 0.00035 in. per in. of length, or an average load of about 50 tons. In this case, it appears that most of the load was taken by skin friction in the sand fill and in the stratum at El. -40.

With the load of 150 tons on pile C-16, the pile shortened  $13/32$  in., or an average 0.00043 in. per in. corresponding to an average load of about 60 tons. It is again evident that most of the load was taken rather high on the pile. This conclusion that most of the loads are taken by skin friction at high levels is confirmed by the measurements of rebound (Table 5).

Table 5 and Figs. 8 and 9 indicate the effects of driving adjacent piles with piles C-13 and C-16, each loaded to 100 tons. The effect is appreciable, but presumably because the compressible strata had been previously consolidated, not as great as the original settlement.

(c) Capacity of Piles

Under a load of 150 tons, the action of pile C-16 was completely elastic, no measurable permanent set taking place. The total settlement under this load was  $13/32$  in., or 0.41 in.

A rule for safe loads given by the Uniform Building Code is to "observe the point at which, after 24 hrs., the total settlement does not exceed 0.01 in. per ton of a test load and divide by a factor of safety of 2."

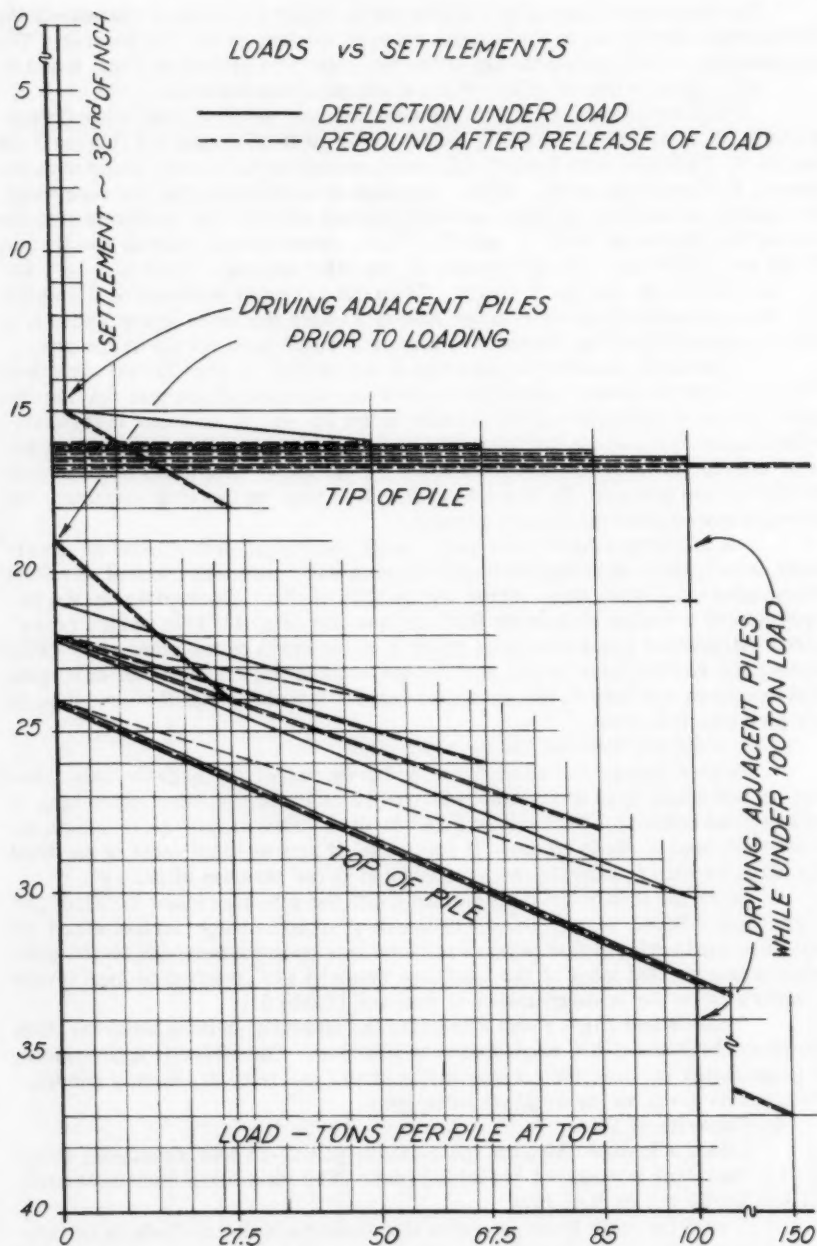


FIG. 8.—PILE C16 - REBOUND CURVE

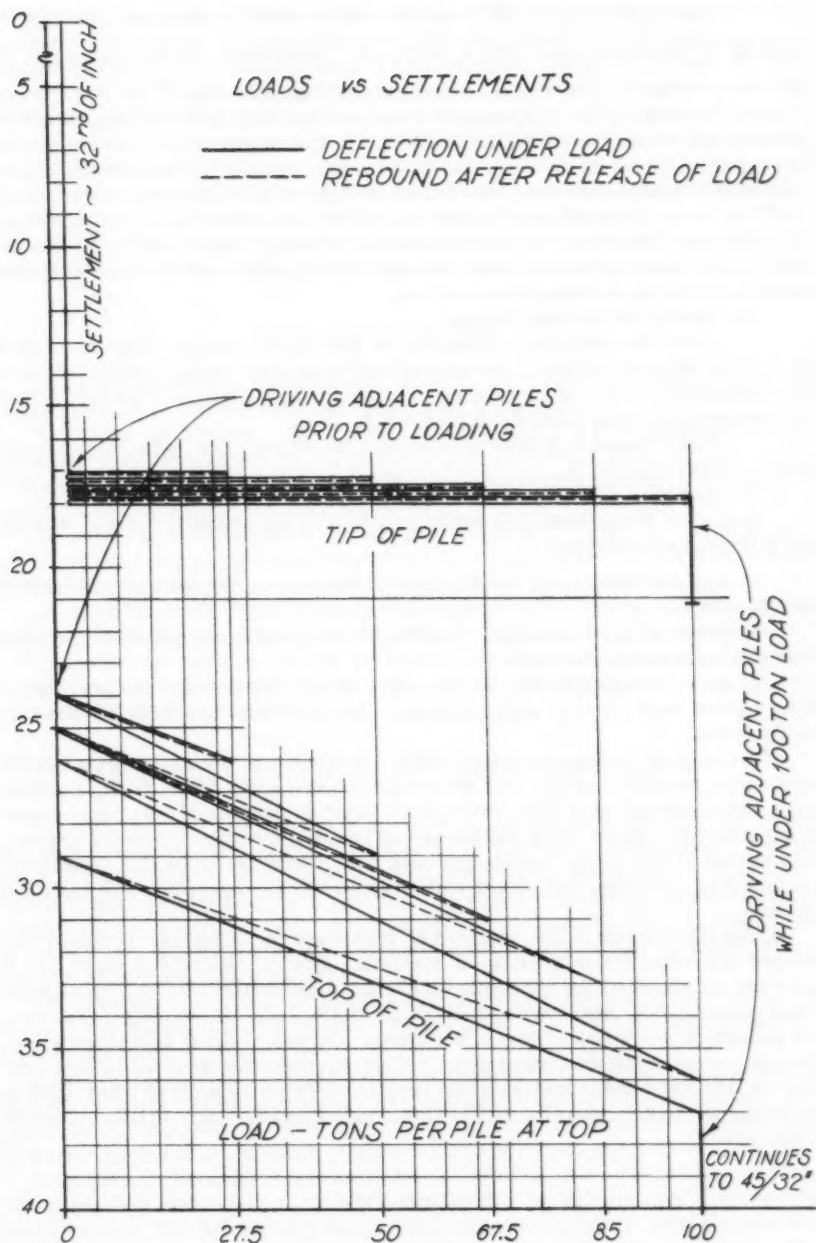


FIG. 9.—PILE C13 - REBOUND CURVE

For a safe load of 75 tons, this requirement would permit total settlement of  $1 \frac{1}{2}$  in. under the 150 ton test load. Therefore, by this criterion, the safe load of these piles would be considerably greater than 75 tons.

In making this statement, it is recognized that load tests on one or two piles do not always develop the settlements that might result from a similar loading on a larger group of piles. In this case, however, the final set of borings demonstrated that the strata below the tips of the piles range from "compact" to "very compact" sand and rock. With the area of 12.5 sq ft per pile, the additional load on these strata will be 4 tons per sq ft for the 50 ton pile load. Under these circumstances, no appreciable settlements will result from the consolidation of these deeper strata.

(d) Radius of Driving Effects

After the observed settlement of previously-driven piles as a result of driving adjacent piles, it became advisable to fix a radius within which no piles should be driven after capping previously-driven piles. This also affects the matter of driving piles for future units.

From Table 4, it may be seen that the effects are negligible at a distance greater than 20 ft.

3) Conclusions

The data from these pile loading tests and the analysis thereof warrant the following conclusions:

a) Any settlements of the building or equipment foundations will be very small.

b) The safe load carrying capacity of the piles is considerably greater than that assumed in the design.

c) As an exception to the above, measurable settlements of the previously-driven piles may occur if additional piles are driven within 30 ft of those already driven.

As a result of the investigations made subsequent to the development of the subsidence problem, permanent instructions were given to the field requiring that all piles should be driven within 30 ft prior to pouring of pile caps. Based on the data assembled, 20 ft should be sufficient. However, in view of experience at this site, a conservative approach was indicated. Since this is entirely a scheduling problem, the conservative approach did not entail any material expense.

During the course of the settlement investigation, a further problem developed regarding the pipe piles. A maximum driving rate of 150 blows per ft had been established for pipe piles with the expectation that practically all would penetrate the hard layers within this limitation. It developed that certain groups of pipe piles failed to penetrate. It was decided that these piles, when in groups, must be forced down. This necessitated working in from the edges of the hard areas and entailed very hard driving on many piles (150 to 200 blows per foot for several feet). This condition exists only in limited areas on the site.

## CONCLUSION

Pile driving difficulties at Port Everglades have led to an investigation of the behavior of a pile under load and of the reaction of a pile to adjacent driving somewhat more extensive than is normally undertaken. It is believed that

the data and analysis presented demonstrate that the effect of negative skin friction brought about by vibratory consolidation due to pile driving can be a damaging construction problem even though adequate design allowance for this phenomenon has been made. The data indicates that the effective radius at this site is about 20 ft. The data further indicates, as might be rationally deduced, that material settlements resulting from this source do not materially affect the load carrying capacity of piles.

At the time of writing, pile driving for this project has been completed and the major loads for Unit No. 1 of the plant have been in place for several months. No material settlements have been experienced. It is hoped that the data presented herein will be of interest and value.

#### ACKNOWLEDGMENTS

The writer takes this opportunity of expressing his appreciation to all those who have assisted during the difficulties described herein.

The client, the Florida Power and Light Company, acting through George Kinsman, Vice-President, H. V. Street, Chief Engineer, B. H. Werry, Construction Superintendent, and their staffs, has cooperated in every possible respect.

The advice of the foundation consultants, Moran, Proctor, Mueser and Rutledge, represented on this assignment by Paul Wentworth, F. ASCE, has been invaluable.

Hercules Concrete Pile Company, Clem Hoppe, President, has cooperated in all possible ways to bring this difficult work to a successful conclusion.

The writer has leaned heavily for technical assistance upon his associates at Bechtel Corporation, particularly Glenn B. Woodruff, F. ASCE; J. George Thon, F. ASCE; and Leslie A. Irvin, F. ASCE.

#### APPENDIX.—EVALUATION OF DIESEL PILE HAMMER

The specification for driving piles at Port Everglades was written to require that the piles be driven with a steam- or air-activated hammer. Shortly after commencing the work, the piling subcontractor proposed that he be allowed to use a diesel hammer. The greater speed, ease of operation, and fuel economy of the diesel hammer made its use appear quite attractive. There appeared to be a potential saving of both money and time. Consequently, a program of evaluation was undertaken.

The hammer being used on the site was a Vulcan OR, energy rating 30,000 lb-ft. The diesel hammer proposed was a Link-Belt No. 520, which also has an energy rating of 30,000 lb-ft.

It was recognized that evaluation of this hammer would entail considerations both of capacity and of reliability. To evaluate capacity, driving tests were made. Both complete driving of adjacent piles and alternate driving on the same pile were performed. Results of these tests are shown in Table 6.

Regarding reliability of this machine, no solution was reached. It is recognized that this hammer can operate while delivering considerably less than its rated energy. No satisfactory visual or audible check on its performance was presented.

*Conclusion.*—Results of comparative driving show that the diesel hammer tested is roughly equivalent to the Vulcan OR hammer in capability. If satisfactory performance checks can be devised, it will be an acceptable and highly

TABLE 6.—DIESEL HAMMER DRIVING TESTS

Pile F245			Pile F247					
Diesel	Vulcan	Penetration	Diesel	Vulcan	Pile No.	Diesel	Elev.	Vulcan
24	22	44						
58	23	50			S97	150	86	150
70	24	53			S83	150	82	150
172	80	56			S69	150	83	101
210	188	57			S64	150	83	80
60	84	59	29	21	S52	150	83	112
25	23	65			S44	150	86	180
27	33	70			S32	150	87	225
25	16	75	18	20	S17	150	94	300
64	35	78	19	11	S7	150	92	Refusal
78	46	82			S6	150	92	Refusal
96	56	85	15	21	S2	150	91	Refusal
144	94	88	46	11	S1	150	94	Refusal
90	62	90						
91	80	94			G289	36	67	34
210	96	96			G270	55	67	62
340	442	99			G251	22	88	31
		101	36	51	G252	34	64	32
		106	48	21	G258	55	66	42
		110	43	76	G264	38	67	41
		123	30	31	G266	51	68	88
		129	21	107	G263	67	69	160
		137	49	54	G254	64	69	Refusal
		143	24	16	G259	34	69	Refusal
		150	22	11	G260	44	66	Refusal
		158	46	159	G261	60	69	Refusal
					G262	78	69	Refusal
					G269	80	69	180
Alternate driving with Vulcan OR and Link Belt Model 520					Redriving with Vulcan OR after initial driving with Link Belt Model 520			

advantageous tool. There are, of course, many installations where driving resistance is not used as a measure of pile capacity. For such cases, this diesel hammer may be used to advantage.

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DRAG COEFFICIENTS OF LOCOMOTION OVER VISCOUS SOILS

By Ervin Hegedus<sup>1</sup> and Robert S. Rowe,<sup>2</sup> M. ASCE

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SYNOPSIS

A supersaturated viscous mud overlaying a hard bottom material is often critical to locomotion in many areas. To determine the drag of a wheel or track moving through mud, the familiar concepts of hydrodynamics pertaining to incompressible viscous fluids may be applied. A correlation between theory and experiment is indicated. The basic problem to be solved is one of viscous flow around a partially submerged object. The variation in pressure resulting from the friction drag causes a bulldozing effect in front of the wheel, and a resulting wake behind the wheel. The pressure drag may be reduced by streamlining the wheel which reduces both the amplitude of the pressure wave and the width of the trailing wake. A comparison between various wheel forms has been made and presented in chart form.

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INTRODUCTION

It is often necessary for a vehicle to cross a terrain composed of a supersaturated viscous soil overlaying a hard bottom. At present (1960) the influence of viscosity, as it relates to the drag resistance of vehicles, is not included in the accepted theory of soil mechanics. However, if a rational basis for vehicle design is to be developed, the effects of viscosity and density should

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be considered.<sup>3</sup> Results from experiment and analysis indicate that the theory of fluid dynamics of viscous incompressible fluids is applicable for the determination of the drag resistance of wheels in viscous mud.<sup>4</sup>

The basic problem to be solved is one of viscous flow around a partially submerged object (Fig. 1). In all cases the velocity is small and the velocity pressure is negligible. Variation in pressure resulting from the friction drag causes a bulldozing effect in front of the wheel and a resulting wake behind the wheel. It should be expected that the pressure drag may be reduced by streamlining the wheel, which reduces the amplitude of the pressure wave and which decreases the thickness of the wake behind the wheel. A comparison of various wheel forms, as illustrated in Fig. 2, shows that a streamline wheel has a smaller wake and pressure wave than a tire or rectangular shaped wheel, and thus offers proportionally much less resistance to motion.

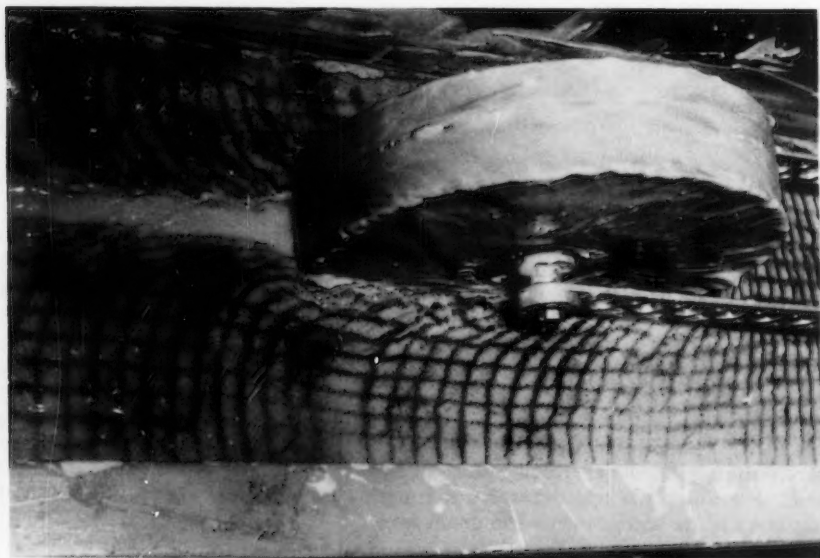


FIG. 1.—WHEEL DRAG AND VISCOUS FLOW FIELD

The fundamental equations pertaining to the fluid dynamics of incompressible fluids are the Navier-Stokes equations. At present there is no general method for the solution of these equations because they are non-linear. There are only a few special cases, however, that can be solved exactly. In every case, assumptions must be made as to the state of the fluid and to the configuration of the flow pattern.

The main mathematical difficulties involved in the solution of the Navier-Stokes equations are due to the fact that the inertia terms are non-linear. Some solutions are possible by assuming that we have incompressible fluids

<sup>3</sup> "Theory of Land Locomotion," by M. G. Bekker, Univ. of Michigan Press, Ann Arbor, Mich., 1956.

<sup>4</sup> "Drag Coefficient in Locomotion Over Viscous Soils, Part 1," by E. Hegedus, Ordnance Tank Automotive Command, Detroit Arsenal, Land Locomotion Lab., Report No. 25, January, 1958.

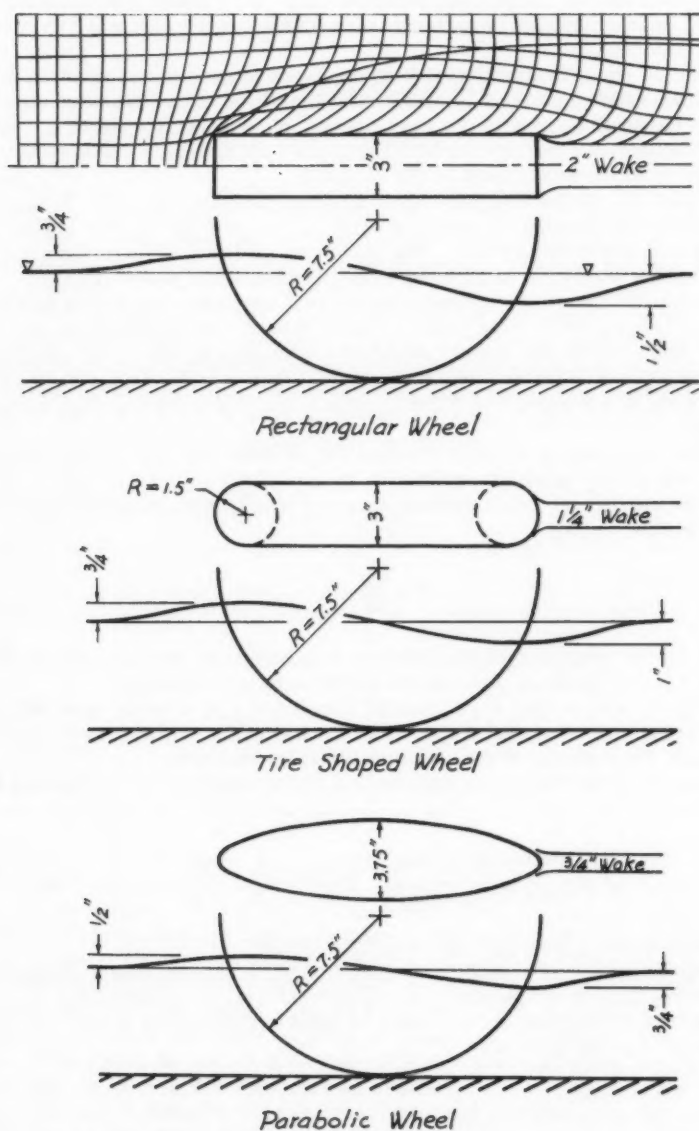


FIG. 2.—PRESSURE WAVE AND WAKE FOR DIFFERENT WHEEL FORMS.

with constant velocity. Additional solutions may be obtained by linearizing the equations, by considering very large viscosities, or by assuming very slow motion.

Because of the mathematical difficulties encountered in the solution of the general differential equations, a major portion of the effort has been directed along experimental lines with the development of empirical equations. Much use has been made of the laws of similitude and dimensional analysis to extend the results of small scale tests to the prototype.

The equations of the boundary layer are approximate and seem to apply to viscous mud even though a turbulent wake is formed behind the wheel as it moves through the viscous fluid. The thickness of the wake depends upon the geometry of the wheel and is reduced by streamlining the wheel (Fig. 2).

The classical theory of hydrodynamics pertaining to ideal fluids has been extensively investigated in the past.<sup>5</sup> Nevertheless, the classical theory fails to explain some of the phenomena associated with real fluids. The ideal fluid is assumed to be friction and incompressible. In order to explain such characteristics as skin friction and form drag on a body, a theory of real fluids is necessary.

Viscosity is known as internal friction and is defined as that characteristic of a real fluid which exhibits resistance to any alteration of its form.<sup>5</sup> Viscosity is the coefficient which relates shearing stress with the velocity gradient in the following way:

$$\tau = \frac{dU}{dx} \mu \dots \dots \dots (1)$$

in which  $\tau$  is the shearing stress between two layers of the fluid,  $dU/dx$  denotes the velocity gradient, and  $\mu$  is the coefficient of viscosity.

From Eq. 1, we see that the tangential force per unit of area, here defined as the shearing stress,  $\tau$ , is proportional to the slope of the velocity curve,  $dU/dx$ , where the constant of proportionality is the viscosity  $\mu$ .

Thus, one may determine the dimensions of the coefficients of viscosity as follows:

$$\mu = \frac{\text{shearing stress}}{\text{velocity gradient}} = \frac{m}{t^2} \frac{L}{L^2} \div \frac{L}{t L} = \frac{m}{t L} \dots \dots \dots (2)$$

in which  $m$  is mass,  $t$  denotes time, and  $L$  is length.

A viscous soil such as a supersaturated mud is, in general, a non-Newtonian fluid as the coefficient of viscosity varies with the rate of deformation.

The coefficient of kinematic viscosity is often denoted by the symbol,  $\nu$ , and may be determined as follows:

$$\nu = \frac{\mu}{\rho} = \frac{m}{\rho t L} \div \frac{m}{L^3} = \frac{L^2}{t} \dots \dots \dots (3)$$

The kinematic viscosity,  $\nu$ , is important where forces are due mainly to viscous and inertia effects.

<sup>5</sup> "Drag Coefficients of Locomotion over Viscous Soils, Part II," by E. Hegedus and R. Rowe, OTAC, Detroit Arsenal, Land Locomotion Lab., Report No. 54, July, 1959.

For ready reference, some of the typical values for coefficient of viscosity, density, and kinematic viscosity for various materials are presented in Table 1.

### SIMILITUDE AND DIMENSIONAL ANALYSIS

Since it is extremely difficult and frequently impossible to solve the Navier-Stokes equations for viscous fluids, it is often convenient to determine a series of relations that exist between various conditions by using other techniques, such as the law of similitude and dimensional analysis. The major effort in fluid dynamics has been along experimental investigations to determine the coefficients that permit one to compute the desired relations by use of empirical equations.

It was first determined by Osbourne Reynolds that dynamic similarity will exist when alterations of the units of length, time, and mass transform the differential equations and the boundary conditions, in one case, into those of another case so that the equations completely coincide.<sup>6</sup> By equating the coefficients of the similar differential equations, various non-dimensional parameters pertaining to identical flow fields may be obtained.

TABLE 1

Material	$\rho$ , $\frac{\text{lb sec}^2}{\text{ft}^4}$	$\mu$ , $\frac{\text{slugs}}{\text{ft - sec}}$	$\nu$ , $\frac{\text{ft}^2}{\text{sec}}$
Air	0.00236	$0.0377 \times 10^{-5}$	$16.0 \times 10^{-5}$
Water	1.97	$2.13 \times 10^{-5}$	$1.08 \times 10^{-5}$
Oil (SAE-40)	1.78	$203.2 \times 10^{-5}$	$114.0 \times 10^{-5}$
Mud (typical)	2.4	$14,400 \times 10^{-5}$	$6,000 \times 10^{-5}$

Another important method of determining the relationship between the model and the prototype, similar to the laws of dynamic similitude, is to apply dimensional analysis which indicates that the physical content of any theory must not depend on the units that are chosen for calculations. Thus, it is possible to use this technique to obtain parameters characterizing the flow without even considering the differential equations which govern the problem in question. The  $\pi$ -theorem is the basic theorem on which applications of dimensional analysis are based. By use of the  $\pi$ -theorem, the dimensionless quantities which characterize the viscous flow may be obtained.

In viscous, laminar, incompressible flow there are five important variables:<sup>7</sup> length, velocity, density, force, and viscosity. These are three fundamental units: length, time, and mass. It is, thus, possible to derive the non-dimensional quantities, called  $\pi$ -groups, in terms of the fundamental units. The first non-dimensional  $\pi$ -group is the drag coefficient  $C_F$  which is used for

<sup>6</sup> "Fluid Mechanics," by Vennard, John Wiley and Sons, New York, 1957.

<sup>7</sup> "Viscous Flow Theory," by Pal, Van Nostrand Co., Princeton, New Jersey, 1956.

most engineering problems where:

$$\pi_1 = C_F = \frac{F}{\rho U^2 L^2} \dots \dots \dots (4)$$

in which  $F$  is a force indicating lift, drag, thrust, or skin friction,  $\rho$  denotes density,  $U$  is the velocity, and  $L$  represents the characteristic length.

The second non-dimensional  $\pi$ -group pertaining to viscous drag is equal to the reciprocal of the Reynolds number and is indicated by

$$\pi_2 = \frac{\mu}{\rho U L} = \frac{1}{R_N} \dots \dots \dots (5)$$

where  $\mu$  is the coefficient of viscosity, and  $R_N$  is the Reynolds number  $= \rho U L / \mu$ .

The Reynolds number is the most important parameter in fluid dynamics of viscous flow and represents the ratio of inertia force to viscous force. When the Reynolds number is small, the viscous force is predominant and the effect of viscosity is only important in the narrow region of the boundary layer. The first dimensionless quantity is a function of the second dimensionless quantity,  $\pi_1 = f(\pi_2)$ ; that is, the force coefficient is a function of the Reynolds number and is indicated by

$$C'_F = \frac{F}{\rho U^2 L^2} = C_F \frac{\mu}{\rho U L} = \frac{C_F}{R_N}$$

or

$$C_F = \frac{F}{\mu U L} = R_N \frac{F}{\rho U^2 L^2} \dots \dots \dots (6)$$

in which  $C_F$  is a force coefficient used for small Reynolds Numbers,  $R_N$ , and slow motion. The viscosity  $\mu$  of the fluid offers resistance to any change in form. This shearing resistance causes a pressure differential to exist between the front and back part of the wheel as it moved through the viscous fluid, as shown in Fig. 2. The total drag acting on an immersed body, is the sum of the pressure drag and the friction drag. The pressure,  $p_s$ , results from a difference in fluid elevation ahead of and behind the wheel. The friction drag results from the shear stress on the wetted surface.

The pressure drag may be obtained by use of

$$D_p = \int_s \left( p_s + \frac{1}{2} \rho U^2 \right) dA_1 \dots \dots \dots (7)$$

in which  $dA_1$  is an element of the projected area in the direction of motion.

The effect of viscosity which produces resistance to the sliding of fluid layers is called a friction drag,  $D_f$ , and is equal to the following equation:

$$D_f = \int_s \tau dA_2 = C_F \rho \frac{U^2}{2} A_2 \dots \dots \dots (8)$$

in which  $\rho$  is the density of the fluid,  $U$  denotes the velocity,  $A_2$  is the wetted area, and  $C_F$  represents the frictional coefficient of drag.

The total drag on a body is the sum of the friction drag and pressure drag, and may be computed by

$$D = D_f + D_p = \int_s \tau dA_2 + \int_s \left( p_s + \frac{1}{2} \rho U^2 \right) dA_1 \dots\dots\dots (9)$$

which may be reduced to the following approximate equation:

$$D = \tau A_2 + \frac{\gamma}{2} (h_1^2 - h_2^2) b + \frac{1}{2} \rho U^2 A_1$$

or

$$D = \frac{2 \mu U}{\delta} A_2 + \frac{\gamma}{2} (h_1^2 - h_2^2) b + \frac{1}{2} \rho U^2 A_1 \dots\dots\dots (10)$$

in which  $\delta$  is the boundary layer thickness,  $A_2$  denotes the wetted surface,  $\gamma$  is the specific weight,  $h_1$  represents the elevation ahead of the wheel,  $h_2$  is the elevation behind the wheel, and  $b$  denotes the characteristic width of the wheel. Eq. 10 is useful for computing the total drag when all necessary quantities have been measured.

The total drag,  $D$ , is usually obtained from experiment, and the drag coefficient,  $C_D$ , determined as a function of the Reynolds Number,  $R_N$ , as follows:

$$C_D = \frac{D}{\rho \frac{U^2}{2} A_2} = \frac{C}{R_N} \dots\dots\dots (11)$$

The measured values may be plotted and used at future times for the solution of dynamically and geometrically similar problems.

## TEST PROCEDURES

The test material consisted of a mixture of volclay and water. Volclay is a special kind of bentonite clay and may be obtained in either powder or granular form. A volclay water mixture is a non-Newtonian pseudo-plastic material. The graph of Fig. 3 shows kinematic viscosity in  $\text{ft}^2/\text{sec}$  as a function of the velocity gradient in revolutions per minute with density in  $\text{slugs}/\text{ft}^3$  as the parameter.

The tests performed with this material were run at values of Reynolds numbers between 0.1 and 1.7. Fig. 1 shows the deformation of the fluid surface ahead of and behind the rolling wheel. In order to investigate the wheel drag in viscous soils, a special preliminary apparatus was built, Fig. 4, which recorded the total drag of the wheel as it moved through the viscous fluid. The mechanical function of the apparatus was as follows.

The wheel (1) being tested rolled on the bottom of the soil bin (2) which measured 12 ft long, 15 in. wide and 15 in. deep by movement of the carriage (3) on the rail (4). The carriage had a strain gage (5) electrically connected to a Brush magnetic recorder which automatically recorded the motion resistance.

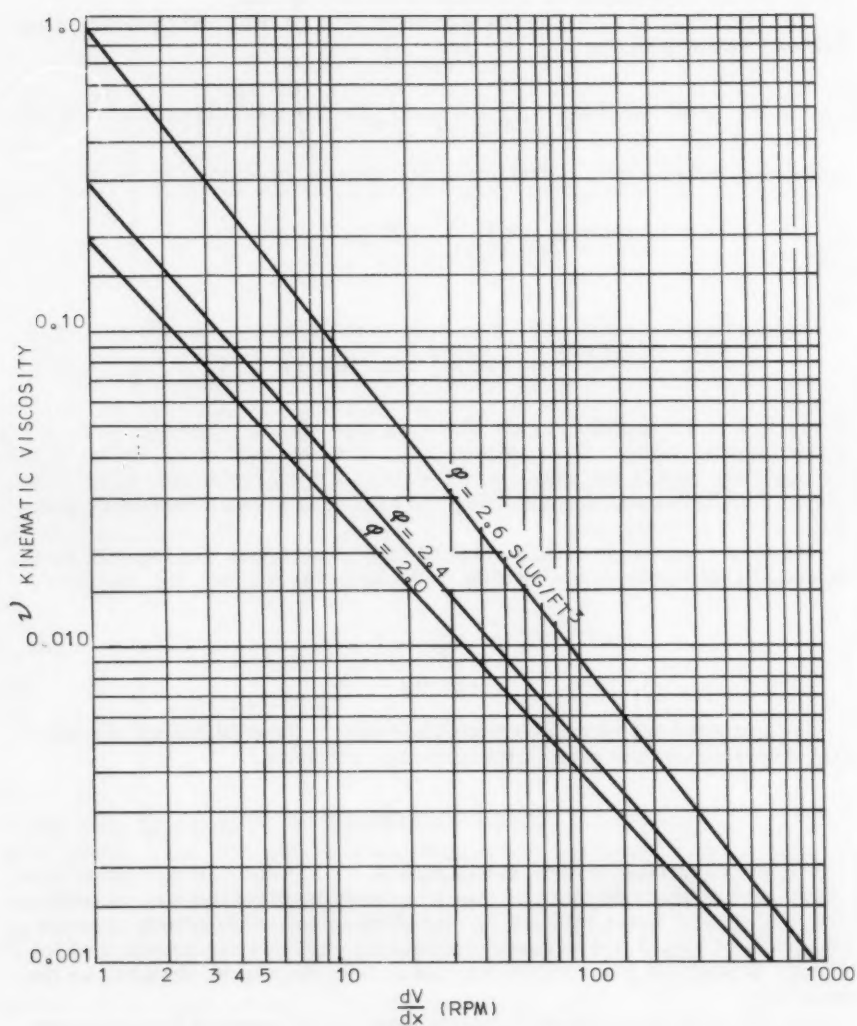


FIG. 3.—KINEMATIC VISCOSITY AS A FUNCTION OF THE VELOCITY GRADIENT WITH THE DENSITY OF VOLCLAY AS THE PARAMETER.

The carriage was moved by a drive mechanism (6) which had a variable transmission from 0 to 15 ft per min. At the end of the soil bin, and limit switch (7) stopped further movement of the wheel. Tests were performed on wheels with different diameters and different widths.

### RESULTS

The results are presented in chart form and show the drag of various wheels in the viscous mud as a function of velocity or Reynolds Number. Fig. 5 is a typical graph showing the total drag in pounds as a function of the velocity in feet per minute for the different wheel types. A comparison of the total drag of the various wheel types shows that the tire-shape wheel has about 30% less drag than the rectangular wheel.

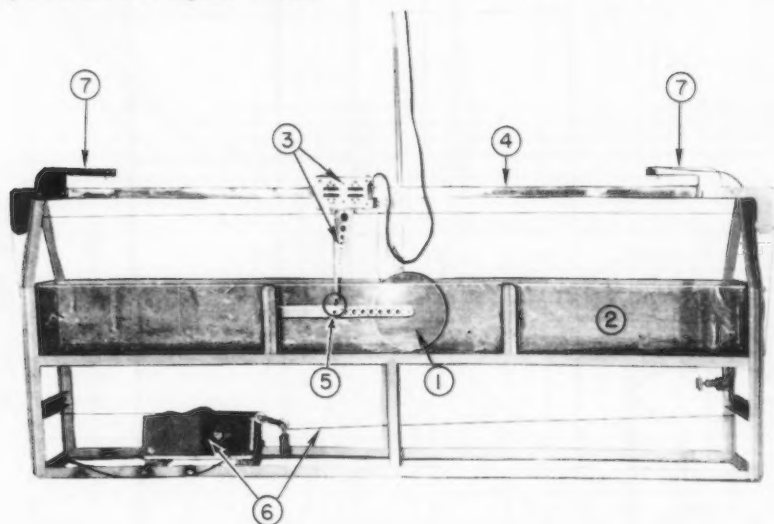


FIG. 4.—TEST APPARATUS

The decrease in drag is due to streamlining with a corresponding shortening of the thickness of the wake behind the wheel. A comparison of the flow field together with the thickness of the wake for the various wheels is shown in Fig. 2.

By use of the measured values of the total drag, the coefficient of total drag,  $C_D$ , was determined as a function of the Reynolds Number,  $R_N$ , by use of Eq. 11. Empirical equations of this type are predominant in hydro-dynamics and permit the evaluation of drag in geometrically similar flow fields. The solution of this equation is usually presented graphically as in Fig. 6, where the coefficient of drag,  $C_D$ , as a function of the Reynolds Number,  $R_N$ , for the various wheel types may be obtained. As a matter of interest, Stokes solution for a sphere completely submerged is also shown in Fig. 6.

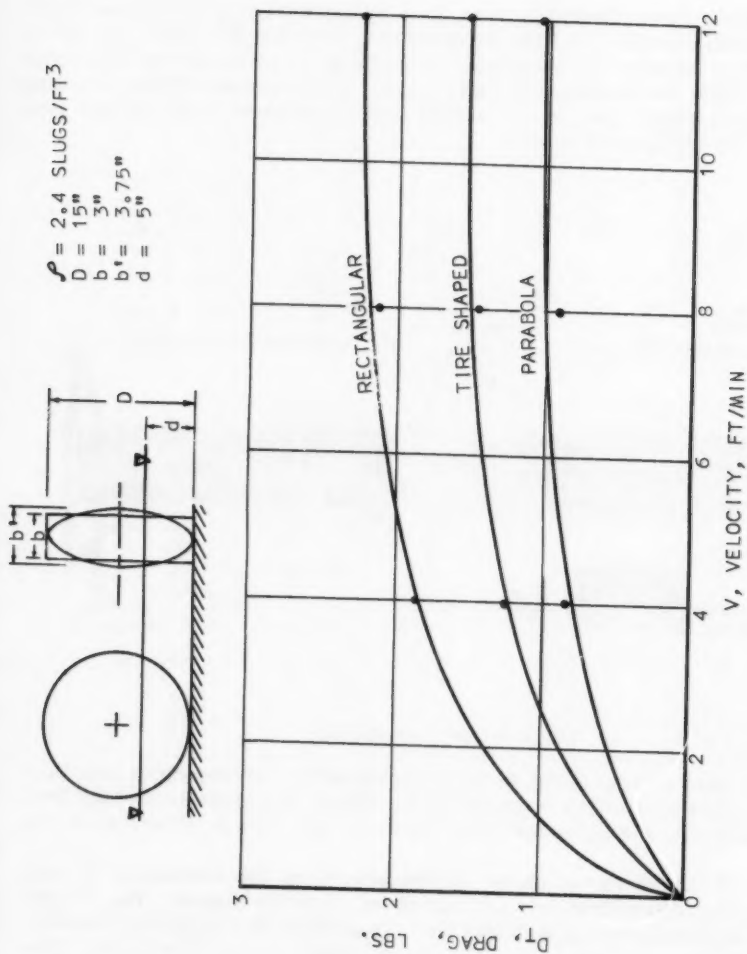


FIG. 5.—TOTAL DRAG AS A FUNCTION OF THE VELOCITY WITH THE WHEEL SHAPE AS THE PARAMETER

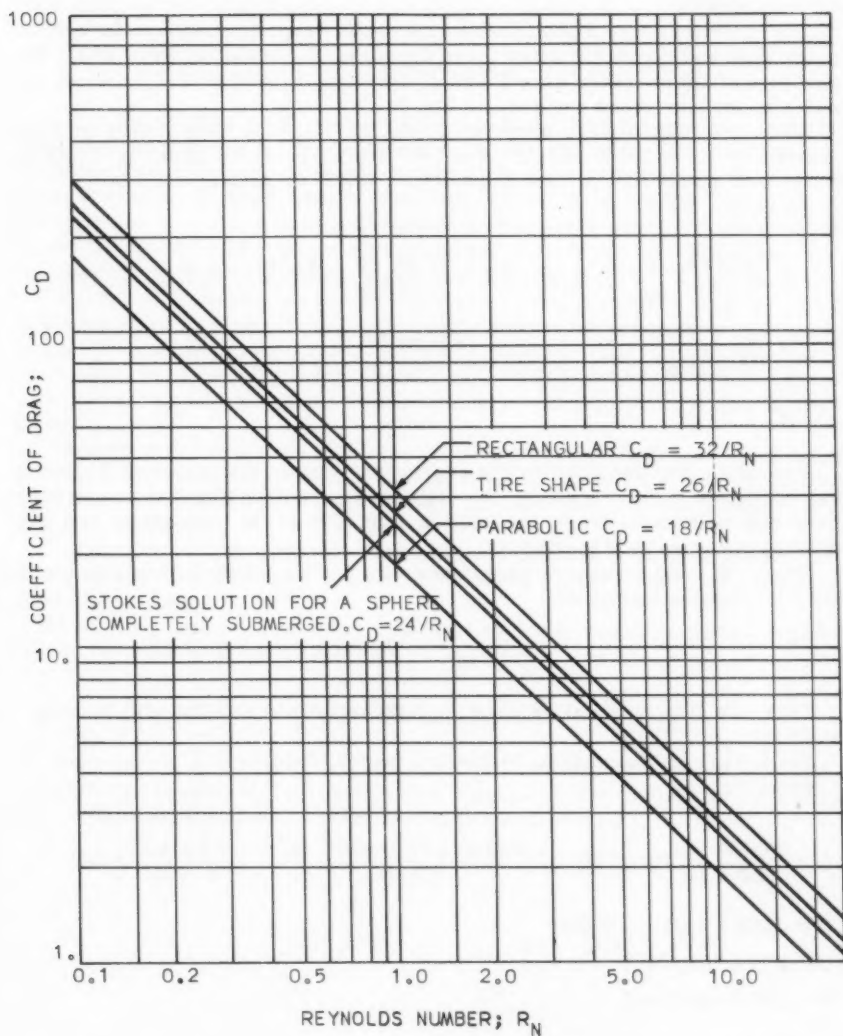


FIG. 6.—DRAG COEFFICIENTS AS A FUNCTION OF THE REYNOLDS NUMBER FOR DIFFERENT WHEEL GEOMETRIES ( $\rho = 2.4$  SLUGS/FT<sup>3</sup>)

## CORRELATION BETWEEN THEORY AND EXPERIMENT

In order to illustrate the agreement between theory and experiment, a typical test was run under controlled conditions with accurate measurements of all parameters.

Fig. 1 shows a typical test with a grid system superimposed on the surface of the fluid so that the boundary layer thickness,  $\delta$ , could be measured. The elevation of the pressure wave ahead of the wheel,  $h_1$ , and behind the wheel,  $h_2$ , was also measured and recorded in Fig. 2.

Measured values were substituted into Eq. 10 and are duplicated here for reference:  $U = 4$  ft per min,  $d = 5$  in.,  $b = 3$  in.,  $A_1 = 0.107$  sq ft,  $A_2 = 1.11$  sq ft,  $\mu = 0.57$  lb-sec/ft<sup>2</sup>,  $\rho = 2.4$  slugs/ft<sup>3</sup>,  $h_1 = 5.75$  in., and  $h_2 = 3.5$  in.

From Eq. 10

$$D = \frac{2(0.57)}{3.0} \frac{4}{60} 1.11(12) + \frac{2.4 \times 32.2(5.75^2 - 3.5^2)3}{2(1728)} + \frac{(2.4)}{2} \frac{4^2}{60} (0.107)$$

$$D = \begin{array}{ccc} 0.33 & + 1.41 & + 0.00057 \\ \text{Friction} & \text{Static Pressure} & \text{Velocity Pressure} \end{array}$$

$$D = 1.74 \text{ lb}$$

The total computed drag for the rectangular wheel amounted to 1.74 lb and was composed of three factors: the static pressure drag (the most important, about 80% of the total drag); the friction drag (20% of the total drag); and the velocity pressure drag (negligible).

The total drag measured experimentally may be obtained from Fig. 5 and has the following magnitude:

$$D_{\text{exp}} = 1.82 \text{ lb}$$

Thus, the experimental value of 1.82 lb compares closely with the computed value of 1.74 lb.

The following data pertains to the tire shaped wheel:

From Eq. 10

$$D = \frac{2(0.57)}{3.0} \frac{4}{60} (0.9)(12) + \frac{2.4(32.2)(5.75^2 - 4.00^2)(2.74)}{2(1728)} + \frac{2.4}{2} \left(\frac{4}{60}\right)^2 0.097$$

$$D = 0.274 + 1.04 + 0.000515$$

$$D = 1.13 \text{ (computed)}$$

$$D = 1.25 \text{ (measured)}$$

The following data pertains to the parabolic wheel:

$$D = \frac{2(0.57)}{3.0} \frac{4}{60} 0.76(12) + 2.4 \times 32.2(5.5^2 - 4.25^2)(2.1) + \frac{1}{2} (2.4) \left(\frac{4}{60}\right)^2 (0.073)$$

$$D = 0.231 + 0.56 + 0.00039$$

$D = 0.791$  (computed)

$D = 0.78$  (measured)

The foregoing agreement is good and shows the relative influence of the parameter on the total drag for the various wheel types

### CONCLUSIONS

From the results of the work conducted to date, it can be deduced that the laws of fluid dynamics are applicable in the determination of the drag in extremely loose supersaturated soil, and that viscosity is a convenient factor in the characteristics of such soils. Solutions by use of the Navier-Stokes equations for flow around partially submerged objects are desired. Boundary layer theory seems to apply even though a turbulent wake is formed when a partially submerged wheel moves through a viscous fluid.

In order to correlate theory with experiment, it is necessary to measure all of the parameters of the fluid profile in front of and behind a moving wheel to include the pressure wave, wake, and boundary layer. This is necessary in order to be able to compute the pressure drag. The effect of the velocity pressure is very small for viscous fluids with very slow motion and may be neglected.

Variations in tire form, which results in the decreased thickness of the wake trailing a wheel, reduce the drag proportionately. A parabolic wheel offers less drag resistance than a tire-shaped wheel, and both offer less resistance than a rectangular wheel.

In extremely loose supersaturated soils, it is necessary to measure the viscosity in order to determine the drag resistance. A new type viscometer for field use is needed.

In any soil value system the effects of viscosity should be included in the theory for both design and analysis.

### ACKNOWLEDGMENT

The writers wish to acknowledge M. G. Bekker, Director, Land Locomotion Laboratory, Ordnance Tank Automation Command, Department of the Army, under whose direction and support this paper was possible.



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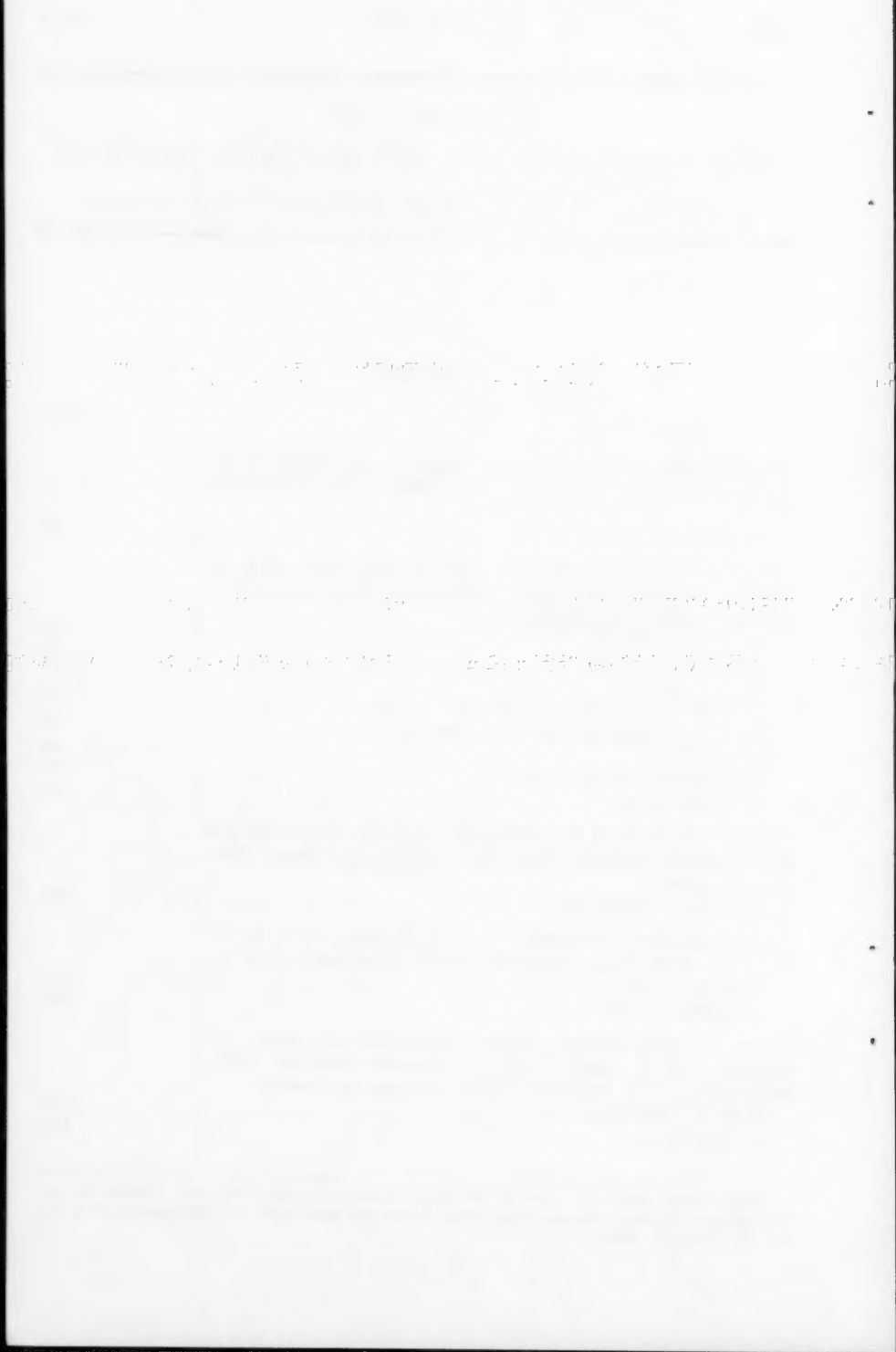
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TRACTIVE RESISTANCE OF COHESIVE CHANNELS<sup>a</sup>

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Discussion by Frank D. Masch, Jr.

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FRANK D. MASCH, Jr.,<sup>1</sup> A. M. ASCE.—The author has presented a very interesting approach to the evaluation of the tractive resistance of cohesive sediments. Investigations of a general nature are always valuable when one considers that most published literature on this problem describes tests that have been performed to verify specific cohesive bed models.

Tests have recently been run at the University of Texas to study the erosion process in cohesive sediments. In these tests, scour was obtained by subjecting the horizontal surface of a cohesive sediment sample to the action of a submerged vertical jet. Although these tests were primarily directed toward the effect of the hydraulic characteristics on the rate of scour, several aspects were similar to those performed by the author.

In these tests, the depth and rate of scour in a given cohesive material were determined as a function of the nozzle velocity and the geometry of the jet. Tests were run on both remolded and essentially undisturbed samples. In all cases it was noted that scour began at points a short distance away from the center of the sample. This effect was reproducible as an initial condition and was independent of the jet velocity and elevation and the nature of the sediment tested. Since it was found that the depth of scour was a function of the logarithm of time, the scour rate was indexed by the ratio of the depth of scour to the log of time. This scour rate index was then found to be a function of the jet velocity, diameter, and elevation. It was also found that when under the influence of fixed jet geometry, a given sediment required a certain minimum jet velocity before any noticeable scour began. The variables influencing the scour rate index were combined into a Reynolds number. The Reynolds number describing the initial conditions was defined as the point of incipient scour. A given sediment was found to have a fixed point of incipient scour irrespective of the jet geometry. Beyond this minimum Reynolds number, however, the influence of nozzle velocity and jet geometry were evident.

It may be well to point out the effect of the nozzle elevation on the scour in cohesive materials. Although changes in nozzle elevation have no apparent effect on the location of the initial scour, elevation changes do effect the nature of the erosion after a very short period of time. The effect of nozzle elevation is explainable in terms of the geometry of the submerged jet. If the nozzle is very close to the sample, the potential core of the jet impinges on the surface of the sample and causes a very localized scour. Under these conditions it was found that a relatively deep narrow scour hole was formed in a very short time. The erosion under these circumstances is not truly representative as the energy of the jet is dissipated by turbulence in a deep scour hole rather than by

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<sup>a</sup> June, 1959, by Irving S. Dunn.

<sup>1</sup> Assoc. Div. of Hydr. and Engrg., Univ. of California, Berkeley, Calif.

causing scour of the surface of the sediment sample. This occurrence was characterized by the relative jet elevation, defined as the ratio of jet elevation to nozzle diameter, of less than 6.5. On the other hand, for  $h/D > 10.0$  the potential core of the jet does not strike the sediment surface and the jet energy is dissipated through shear in causing a wide area to be eroded. It was concluded that in tests utilizing a submerged jet, the ratio of jet elevation to nozzle diameter should be equal to at least 10.0.

These tests at the University of Texas have just recently been completed. The results will probably be submitted for publication at a later date.

COMPRESSIBILITY AS THE BASIS FOR SOIL BEARING VALUE<sup>a</sup>

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Discussions by Kurt H. Siecke; David M. Greer; R. C. Harlan;  
Louis J. Goodman and Charles N. Lee; F. H. Kellogg and  
E. L. Murphree, Jr.; Lev Zetlin; Bramlette McClelland; and James Chinn

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KURT H. SIECKE,<sup>1</sup> F. ASCE.—Whatever may be their individual method of analyzing data and determining design criteria, engineers engaged in foundation design will be greatly interested in this simplified and logical basis for soil bearing values. Present design procedures are often in the category of an art by reason of the manner in which personal judgment and experience enter into foundation design.

This being the case, it is questionable whether the designer always recognizes the theoretical basis for his selection of an allowable bearing value, or is concerned that it may be based on the rupture theory rather than on consideration of settlement. Nevertheless, except in the case of incompressible foundation materials, accepted procedure takes serious account of settlement as well as the comparison of conditions at a site with those previously encountered.

The author bases his reasoning on well-known and generally accepted theory. His decision to use the inverse of the compression ratio, called the bearing capacity index, as a function of values of  $N$  blows per ft obtained from the standard penetration test, is logical for the reason that both the bearing capacity index and values of  $N$  will, thus, be consistently related.

Although Terzaghi and Peck have previously defined the term "significant depth" as that depth within which the load on a footing alters the state of stress in the soil enough to produce a perceptible contribution to the settlement, this definition is not self-evaluating. Therefore, even though Mr. Hough's definition of the "depth of significant stress," as the depth from the upper boundary to the level at which  $p_1 = 10 \Delta p$ , is entirely arbitrary, it has the merit of providing a logical reference point for foundation design. Whether one agrees with the resulting value or not, it appears that the recommended term deserves consideration, if only for the reason that it enables one to arrive at consistent solutions.

The introduction of the allowable pressure index  $K$ , to establish the relationship between allowable contact pressure and the unit weight of the soil, also appears to have a logical purpose. However, it is important in this regard to remember the assumptions which were used in proving the relationship of Eq. 37 ( $p'_c = \gamma K$ ) and to review the process of calculating values of  $K$  as related to the bearing capacity index, the size of footing, the depth of excavation, and the permissible settlement.

This relationship is provided in Fig. 5, and he indicates the importance of

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<sup>a</sup> August, 1959, by B. K. Hough.

<sup>1</sup> Cons. Engr., Portland, Oreg.

<sup>2</sup> "Soil Mechanics in Engineering Practice," by Terzaghi and Peck, Table 22.

such diagrams by reference and by implication. This relationship is the key to the general application of Eq. 37. In fact, the solution of the two simultaneous equations, by which the depth of significant stress is eliminated for footings below the surface and for surcharged footings, would be more time-consuming and indirect than methods based on compression diagrams.

Recognition of this difficulty does not seriously affect the author's argument nor detract from the value of his conclusion. The usual object of settlement calculations is not to determine absolute values of predicted settlements, but to so proportion the footing sizes and load intensities that serious differential settlement will be avoided. If the author's recommended simplified procedures can be followed, consistent results are possible, and this objective will be realized.

However, it is extremely important that knowledge and judgment be used in applying this or any other simplifying procedures in such a field as soils engineering. Aside from observation of the stated assumptions, the designer must recognize situations to which a particular method does not apply. Even the frequently disparaged standard load test has great value in some instances, if only to account for inconsistencies in other data.

On one occasion in the writer's recent experience, an addition was to be made to a large industrial boiler house. The building frame consisted of structural steel and the new steel was framed directly into the existing columns with provision for adjustment during and after erection, often a wise precaution which can be taken at little cost.

Extensive grading operations had removed the original top soil and exposed a layer of dense homogeneous clay having a moisture content of about 20%, a unit weight in place of 105.6 lb per cu ft, and a void ratio of 0.97. Borings to a depth of 30 ft showed the same soil. No penetration tests were made, but a load test made at the time of original construction, on a 1 sq ft area, showed a settlement under a 6,000 lb load of 0.0625 ft after 216 hr. Laboratory tests showed a dry weight of 86.7 lb per cu ft, a liquid limit of 42% on a dry basis, a plastic limit of 22%, and a plasticity index of 20. The soil was described as a dense inorganic clay of medium plasticity.

Information obtained<sup>2</sup> indicated that the ultimate bearing capacity should be about 7 tons per sq ft, and that the safe bearing capacity (presumptive bearing capacity) could be at least 2 tons per sq ft. This source also stated that  $N$ , for the standard penetration test, should be at least 15 blows per ft and the unconfined compressive strength 2 tons per sq ft, but the latter information was considered to have little value under the circumstances.

Since the differential settlement between the new addition and the original structure was very important, an effort was made to estimate the total settlement to be expected. For a typical 6-ft-square footing, it was decided that the significant depth would be taken at 10 ft below the footing or 12 ft below the ground surface as regraded. By Newmark's influence chart, the vertical pressure increment at 10 ft depth for the soil was described and a contact pressure of 4,000 lb per sq ft was computed as 600 lb per sq ft.

Using the value of the compression index  $C_c = 0.20$  for ordinary clay, having a liquid limit of 42% calculated by reference to Fig. 133 in the same source, the total settlement was estimated to be 0.741 ft for a mean effective pressure of 2,310 lb per sq ft. This settlement would be excessive considering the resulting deformation of the structural steel frame of the boiler house, and was entirely inconsistent with the results of the bearing test which indicated a probable settlement of 0.04 ft to 0.10 ft at the most, under the design load.

However, the likelihood of heavy precompression under early glaciation is well established by the geological history of the area. Preloading of this kind cannot be evaluated precisely but might conceivably have been as much as 5 tons per sq ft, which greatly exceeds the intended loading. Laboratory tests on precompressed clay are considered to be very difficult and quite unreliable. However, it is generally agreed that settlement of precompressed clay under loads which fall on the recompression curve will probably not exceed 10% of the settlement of similar normally loaded clay.

On this basis, the predicted settlement of the proposed footings would be about 0.07 ft, which would be tolerable. By comparison with the result of the load test, it seemed likely that total settlement under the design load would probably not exceed 0.04 ft or  $\frac{1}{2}$  in. The footings were so designed and no difficulty was or has been encountered.

The importance of settlement in the design of certain footings has long been recognized, even if not to the extent just noted. If the settlement predicted by any method is to be better than an approximation, every effort must be made to obtain sufficient, accurate information from the site investigation and from geological history. It should be assumed that density, void ratio, liquid limit, and penetration resistance within the significant depth will be determined in order to determine soil bearing capacity. Unconfined compression strength should also be found.

In most cases, estimates of probable settlement under load can be only approximate because of the practical impossibility of obtaining adequate site investigation and sampling. This situation places a premium on judgment and experience, and often a man so equipped will reach sound conclusions from field data which may appear quite inadequate. Based on the writer's experience, if soil conditions at a site appear to dictate a very detailed costly program of sampling and laboratory testing, its attractiveness for industrial development is greatly diminished.

Therefore, to the extent that his paper is used as a guide to judgment, and to provide the basis for an inexpensive preliminary evaluation of certain sites and the applicable design procedure, the author makes a valuable contribution to the design of footing foundations. The suggested procedure is a short-cut to final design under some conditions, but unless the limitations imposed by basic assumptions are kept in mind and the data applying to a given site are prepared in the form of diagrams and carefully analyzed design factors, the results will remain approximate.

Perhaps this is as much as we should expect in the present period of coordinating soil mechanics theory and practice. Foundation design has its intensely practical aspects, not the least of which is the need for justifying increased expenditures for site investigation. The above example cites an instance where comparatively inexpensive field and laboratory work proved adequate even though some desirable information was not obtained.

In conclusion, the author has prepared a strong case for the greater need of evaluating soil compressibility rather than shearing strength in site investigation, and his belief, that more attention should, in the future, be given to equalizing settlement due to soil compression than to rupture analysis, is practical and realistic.

DAVID M. GREER,<sup>3</sup> F. ASCE.—The dimensionless bearing capacity index, and the ingenious development of convenient methods of using this parameter for proportioning footings to produce equal settlements, should have a marked effect in clarifying the thinking regarding the determination of soil bearing capacities in actual practise.

Mr. Hough's thesis that the real "bearing capacity" of a soil is far more often a function of its compressibility than of its shearing strength, is, of course, correct. Most engineers have made settlement analyses, based on the results of consolidation tests, to arrive at allowable bearing pressures. Such an analysis is usually undertaken only when, in the soil engineer's judgment, the soil is sufficiently compressible to cause concern about possible settlement, when undisturbed samples of the compressible soil are available or can be obtained, and when the proposed structure is sufficiently sensitive to settlement to warrant the additional work of a settlement analysis.

The procedure developed in the paper provides a logical and convenient method for proportioning footings to produce equal settlements of a predetermined amount, provided the soil beneath all of the footings is uniform in compressibility, and provided, also, that the parameter, the bearing capacity index, is determinable by one means or another.

In a uniform cohesive soil, the bearing capacity index can be determined easily from a consolidation test on an undisturbed sample. Data can be collected from consolidation tests on samples from various depths, and at various densities and moisture contents, within a given geologic formation, so that a value for bearing capacity index within that formation can be selected quickly on the basis of textural classification, depth, and in-place density.

But another soil, with a different geologic history, will not necessarily have even approximately the same bearing capacity index, though its textural classification and in-place density may be similar. The author says, in his final paragraph, that "there appears to be a reasonable prospect of correlating this index with textural classification and in-place density." The writer believes that, for cohesive soils, this is true only within geologically related groups of soils, which have been studied in the laboratory.

Fig. 3 presents curves of bearing capacity index against "standard" penetration resistance. Although Mr. Hough introduces this chart as presenting only the order of magnitude of bearing capacity index for various classes of soil, there is a probability that some engineers will take design values directly from it—in fact, the author suggests this procedure. Considering the differences between, say, sensitive and non-sensitive clays of the same penetration resistance, this procedure seems questionable.

The writer wonders about the basis for the curves of Fig. 3, for cohesionless soils. Laboratory determination of the bearing capacity index for these soils is not feasible at the present state of the science (1960). Large numbers of field observations, on actual structures and/or large-scale field bearing tests, would provide suitable data; but it would be interesting to know what scatter of points occurred; and what distinction may have appeared between data from fluvial, glacial, marine, and aeolian deposits of similar texture. In fact, the use of "standard" penetration resistance to determine an index to compressibility seems very questionable. At best, the "standard" penetration resistance is a poor index to shearing strength. That it should turn out to have a uniformly good correlation with a compressibility index seems doubtful. It is

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the writer's opinion that a continuous static cone penetration test, as used in the Netherlands and Sweden, offers much better promise of correlation with shearing strength of cohesionless soils, and possibly with compressibility, as well.

R. C. HARLAN,<sup>4</sup> A. M. ASCE.—The basic idea presented by Mr. Hough is that compressibility should be the chief basis for establishing soil bearing values, and that foundations should be sized so as to limit or eliminate differential settlement.

Both the establishment of allowable bearing values and the design of foundations are important functions of the foundation engineer. The writer does not agree with the author that usual practice is to proportion spread foundations so that the contact pressure does not exceed the "allowable bearing value of the soil." Certainly the experienced foundations engineer, when faced with an important structure on a compressible subgrade, designs the foundations with more thought than is inferred from the author's paper. If such is not the case, then much of Mr. Hough's effort, as well as that of other fine teachers, toward providing a better understanding of foundation behavior, has been wasted.

Some discussion of the term "allowable bearing value" appears desirable, since in several instances the author has used this term where the words "design bearing pressure" might be more descriptive. An allowable bearing value, which may be defined as the maximum foundation contact pressure to be imposed on a given site, is normally established as a part of a foundation investigation. Selection of the allowable bearing value is a matter for considered judgment, often supplemented by test borings or pits, and sometimes by an analysis of the physical properties of the subgrade material. The selection is based on factors such as; the strength and compressibility of the subgrade material; type and extent of structure; topography of the site as it affects cutting, filling, and foundation elevations; and, type and magnitude of loading. The selection is sometimes tempered by the results of rupture analysis based on the results of shear tests, but such analysis should never be the sole criteria for selection of an allowable bearing value. In practice, the allowable bearing value provides an upper limit for the design foundation contact pressures which may then be selected, or established by proportioning the footings, so as to minimize differential settlements.

The shortcomings of several building codes with respect to the design of foundations and determinations of allowable bearing values has been aptly discussed. It must be recognized that many older building codes contain provisions based on long-established local practice and are intended solely to provide for the public safety. However, it should not be assumed that all building codes are uniformly obsolete. One of the most widely used building codes is the Uniform Building Code of the International Conference of Building Officials. A comparison of this code on some of the points discussed by the author may be of interest. Section 2805 of the UBC permits deviation from the table of Allowable Soil Pressure, No. 28-B, "after performance of a special soil investigation by an agency acceptable to the Building official." In practice such an agency is a qualified soils and foundation engineer. The UBC permits increases in bearing values up to 20% per ft of depth in excess of minimum footing depth. Increases for footing widths greater than minimum are provided

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for granular soils. Section 2806 of the UBC requires that "Footings shall be designed to minimize differential settlement."

Where local building codes are obviously in need of revision, it is incumbent on local engineers to initiate a change through local committees in cooperation with building officials. Where changes are recommended it must be established that they are in accordance with sound practice and afford adequate protection for the public safety without prejudicing technical advances. Where building damage results from differential settlement, the cause of the settlements can usually be traced to inadequate construction, or to widely varying subsurface conditions which were not discovered during the subsurface investigation. It would be interesting to learn of building damage which can be directly attributed to a failure to proportion footings so as to minimize differential settlement.

In developing data for foundation design based on compressibility, Mr. Hough has made use of well known and established equations which are subject to equally well established, but perhaps less often considered, controlling conditions. In attempting to develop sound procedures from such equations, the conditions which control the use of the equations must always be kept well in mind. The author has developed a term which is the reciprocal of the compressibility ratio and designated the "bearing capacity index." Because of the nature of this "index," involving the compression index and the initial void ratio of a soil mass, few designers will grasp its significance and limitations. It is undesirable to use tables such as Table 3 in a handbook-type procedure without a full understanding of the conditions which should control their use.

It appears that for the theoretical development of the equations, it has been assumed that subgrade masses are all normally consolidated soil deposits. This assumption is inherent in the use made of "initial stress" and "compression index." Allowance for conditions other than that assumed are apparently made by utilizing the unsubstantiated data of Table 3. Since, in the writer's experience, some overconsolidation is more often encountered in natural soil deposits than normal consolidation, a knowledge of the past stress history of the soil mass is of great importance in properly designing foundation structures so as to minimize differential settlements.

Consider briefly the case of a moderately overconsolidated subgrade mass. It can be readily seen, that for two isolated footings of greatly different loading, the author's procedures would lead to a relatively high bearing pressure, beyond the preconsolidation level, for the small footing. The larger footing, however, would not reach pressures beyond the prestress level, and would, therefore, settle less than the smaller footing. The general effect would be to reduce the amount of total settlement, but to induce a differential settlement with the smaller footing settling more than the larger. This effect would be lessened or increased, depending on the compression index of the soil.

In consideration of foundations in a laterally extensive excavation, Mr. Hough has developed equations for a case which is analogous to an overconsolidated soil condition. In analysis involving overconsolidation, the amount of overconsolidation, expressed in feet of overburden, would be equal to the depth of excavation in the author's case.

The author has presented an interesting and worthwhile paper and should be congratulated on his succinct treatment of the selection of design foundation pressures for various depths and widths of footings. While there is merit in the proposed design procedures, the reliance on such a welter of charts and graphs as would be necessary to cover all possible foundation conditions, may be questioned. Sound foundation engineering requires an understanding of all

the factors inherent in a given situation, and mechanical procedures are best avoided where compressible subgrades must be properly considered. If the procedures are to be more properly utilized, they should be adapted to determine and consider the past stress history of the subgrade mass.

The basis for the procedures is a determination of a "bearing capacity index" for various types of soil which has been apparently correlated with standard penetration tests to reflect initial void ratio. The use of standard penetration tests as a basis for foundation design based on compressibility, seems incongruous. It almost appears as though the author is suggesting a step both forward and backward at the same time. In recent years much has been accomplished in improving both field and laboratory techniques and reducing the time and cost of obtaining and testing undisturbed samples for more conventional settlement analyses. Sufficient evidence has not been presented to show that the author's methods are of sufficient accuracy to justify the possibly reduced cost for even routine site investigations. The information which would be obtained by a geologic reconnaissance of the site, test borings, and the taking and testing of a few undisturbed samples with attendant settlement analyses, is more complete and useful.

Mr. Hough's plea for more research on field behavior of foundations is timely and well worthwhile. It is believed that such research will emphasize the importance of past stress history on foundation behavior.

L. J. GOODMAN,<sup>5</sup> M. ASCE and C. N. LEE,<sup>6</sup> A. M. ASCE.—A timely discussion of one of the most important problems in structural design has been presented—that of determining the allowable bearing capacity of a given soil for foundation-design purposes. The writers agree with the author's statement that "current methods for determining allowable bearing values differ widely in nature from the crudest empirical procedures, to various forms of engineering analysis." The topic is in need of review.

Bearing capacity has been defined in many different ways, but the meaning most frequently given to this term by structural engineers is as follows: Allowable bearing capacity is the maximum pressure that can be transmitted by a structure to the soil which supports it without causing either a shear failure of the loaded soil or excessive settlement, both total and differential. (The factor of safety against soil rupture that is generally used is 2 to 3 and depends on the soil and loading conditions along with size and depth of the foundation.) It should be noted that this definition recognizes both soil shearing strength and compressibility. Therefore, the author's suggestion to use compressibility as the basis for the soil bearing value is not necessarily a new concept, but his suggested approach is rather unique and certainly deserves commendation and further consideration.

Indeed, experience shows that the allowable bearing values for cohesionless soils are generally governed by settlement considerations. The major exception to this is in the case of narrow wall footings, seated at or just below the surface of loose saturated sands where shear may control. On the other hand, the approach generally used today for cohesive soils is to consider the shearing strength in determining the allowable bearing values. If the clay is soft to medium, the settlement is likely to be excessive and must be investigated. If the clay is precompressed, the settlement is likely to be within tolerable limits.

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In any event, a check should be made on the settlement consideration before a final bearing value is selected.

Engineering analysis of soil bearing values on small routine projects is generally confined to a study of soil profiles and standard penetration test data (140 lb hammer, 30 in. fall, 2 in. outside diameter by 1.375 in. inside diameter sampling spoon) from the test boring operation. Considerable data relating soil density, consistency, and strength to the standard penetration test have been collected and can be analyzed by a qualified person to estimate bearing values. The writers have found this to be a reasonable approach as long as both strength and compressibility are considered in the analysis.

Experimental determinations of strength and compressibility characteristics should be made when possible, particularly on large important projects. Undisturbed samples are necessary for these determinations so that no distinct advantage is apparent in determining compressibility in lieu of strength characteristics in so far as the scope of the site investigation is concerned, as implied. Actually, larger diameter undisturbed samples should be taken for consolidation tests and more time is generally required for settlement studies than for soil-rupture considerations. However, it is certainly agreed that adequate data must be obtained from every site exploration program to afford a basis for a proper study of the subsoil conditions and a safe and economical foundation solution.

The shearing strength of cohesionless soils is a function of internal friction which, in turn, is dependent on the density of the soil and, therefore, can be estimated from boring and classification data with a sufficient degree of accuracy. The shearing strength of cohesive soils is much more complicated but is generally taken equal to the cohesion in practice. Cohesion can usually be determined quickly and inexpensively from unconfined compression tests on undisturbed samples or estimated from boring data on small routine projects. It would now be desirable, as Mr. Hough points out, to have a better correlation between compressibility characteristics and soil data normally obtained from every site investigation, such as resistance and classification data. In many cases, the compressibility of soft clays can be evaluated from Atterberg limit and natural water content data. However, it is recognized that more data relating compressibility to soil characteristics (easily obtained from even routine investigations) are necessary.

Building codes and engineering handbooks contain tables showing allowable soil pressures for different types of soil. These serve a useful purpose when proper judgment is exercised but, unfortunately, these tables usually divert the attention of the designer from the fact that the bearing capacity depends on many factors other than soil description. It is generally recognized that it is impossible to assign allowable bearing values to various soils which can be used under all circumstances for small and large footings alike, at all depths below the ground surface, and for any degree of density and saturation of the soil. Also, many of the tables in use by cities in the United States are copies of those of New York, Chicago, or other cities actively interested in this problem, and they do not offer any hint regarding the origin of the values, or any other explanation that may be pertinent. The foregoing is mentioned to emphasize that many building codes need modernizing. It is the writers' opinion that this area also deserves immediate attention, particularly since compliance with these codes is mandatory in many parts of the United States, as the author points out.

Structural engineers do not agree on how much differential settlement can be tolerated, but experience indicates that it can vary from  $\frac{1}{2}$  in. to 2 in. between columns spaced approximately 20 ft apart, depending on both the type of building and construction material used. It is certainly agreed that it would be desirable to design all footings for equal settlement. However, from a practical viewpoint, it must be recognized that the lack of homogeneity in the various soil types would command too elaborate a subsurface investigation to approach this objective. It is also questioned whether fine distinctions between settlement predictions, such as shown by the author in Fig. 5, are in accord with the assumptions of consolidation theory, stress transmission theories, and actual field conditions.

It is generally recognized that the settlement of a loaded area comes from three sources:

1. Consolidation as a result of the decrease in the volume of the loaded soil caused by the gradual expulsion of water from the voids.
2. Elastic compression caused by a lateral bulging of the loaded soil and which may take place without a change in volume.
3. Plastic flow which is characterized by a lateral displacement or progressive shear failure of the soil.

Settlement predictions based on the results of soil tests and consolidation theory as suggested by Mr. Hough have been used and found to be sufficiently reliable if the subsoil contains one or more layers of normally loaded clay; a soil is said to be normally loaded if it has never been acted upon by vertical pressures greater than those existing at present. If the clay is preloaded, the estimates are less reliable, giving predicted settlements that are larger and, therefore, more conservative, than those that may actually develop. Settlements due to elastic compression are usually minor as compared to those produced by consolidation and, as the author states, are generally ignored for settlement analysis. Plastic flow may be prevented by using an adequate factor of safety against shear failure in the soil.

A method sometimes used to estimate settlements and allowable bearing capacity is the use of load tests. The danger of these procedures, due to the influence of footing size, is illustrated by Mr. Hough and, therefore, emphasizes the fact that proper performance and interpretation of load tests are imperative for adequate design. Unfortunately, the load test as usually conducted is a waste of time and money and may even result in a dangerous foundation situation.

It is interesting to note the difference in the allowable soil pressures proposed by the author and those proposed by Karl Terzaghi, Hon. M. ASCE and Ralph Peck, F. ASCE for sandy soils.<sup>7</sup> Fig. GL1 illustrates this difference for a square footing seated at a depth of 2 ft below ground surface in a dry sandy soil with an average  $N$ -value of 20. It should be mentioned here that the initial expressions for the author's Eqs. 28 and 30 were used in arriving at his curve in Fig. GL1. Comparisons for other  $N$ -values and surcharge conditions have been made but time and space preclude their presentation. It is noted that settlement controls at a smaller footing width as the surcharge increases.

Again it is felt that Mr. Hough is worthy of much commendation for his discussion of the compressibility approach to this important topic on bearing capacity. It is felt that the major difference between the two methods discussed

<sup>7</sup> "Soil Mechanics in Engineering Practice," by Karl Terzaghi and Ralph Peck, John Wiley & Sons, 1948.

herein is in the interpretations of representative values from stress-strain curves. However, it is the writers' opinion that if the author's paper stimulates the necessary interest in this problem, it has accomplished a much-needed objective.

F. H. KELLOGG,<sup>8</sup> F. ASCE and E. L. MURPHREE, Jr.<sup>9</sup>—This paper demonstrates convincingly that design of footings for safety against failure by rupture does not, of necessity, guarantee that there will be no undesirable differential settlement. This was also demonstrated by Skempton<sup>10</sup> on the basis of an analysis of elastic settlement. Nevertheless, use of factors of safety against rupture of 2 to 3 have been used for a long time as the sole criteria for determination of allowable bearing capacity. One may well ask why, if this practice is

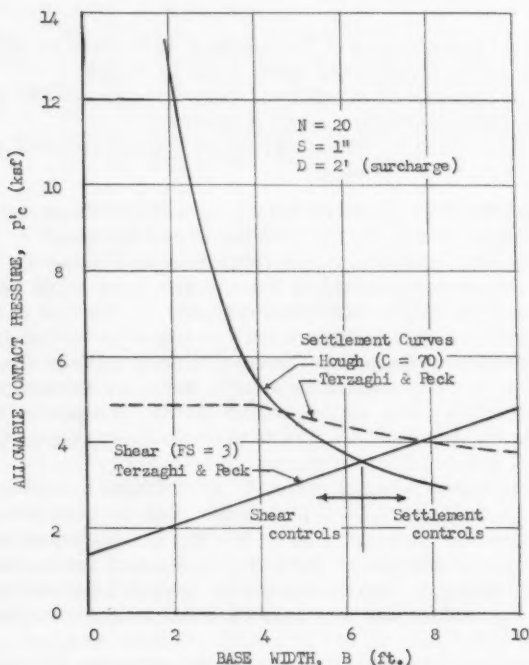


FIG. GL1.—COMPARISON OF SETTLEMENT AND SHEAR RESTRICTIONS ON BEARING CAPACITY SQUARE FOOTINGS SEATED ON A SANDY SOIL

inadequate, more foundation trouble has not been reported. The fact that the designer does not always hear about settlement cracks and other minor ills that sometimes develop in structures may offer a partial explanation. Of more significance, however, is the increasing tendency toward use of designs that

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<sup>9</sup> Dir., Computer Center, Univ. of Miss., University, Miss.

<sup>10</sup> "The Bearing Capacity of Clays," by Skempton, Building Research Congress, 1957, pp. 180-189.

demand much closer settlement tolerances than were formerly required. Longer spans, more glass, thin-shell concrete, and close connections between units in chemical and mechanical plants, all require rigid limits on differential settlement.

It would be most helpful if the author could give statistical data on the curves in his Fig. 3, such as the number of tests, correlation coefficient, and standard error of estimate. An empirical correlation has been suggested between the slope of the virgin compression curve and the liquid limit which is expressed by the equation

$$C_c = 0.009 (LL - 10) \dots\dots\dots (KM1)$$

where LL represents the liquid limit in percentage.<sup>11</sup> When this is modified for low water-plasticity ratios, it gives a fairly good correlation for a wide variety of soils, although individual deviations can be large. In these cases, however, corresponding values of  $C$  would be much less than those shown in Fig. 3, leading the writers to ask whether the higher values in that figure represent recompression curves.

In analyzing settlement, it is necessary to make one of two mutually exclusive assumptions. The one more commonly encountered, which was used by

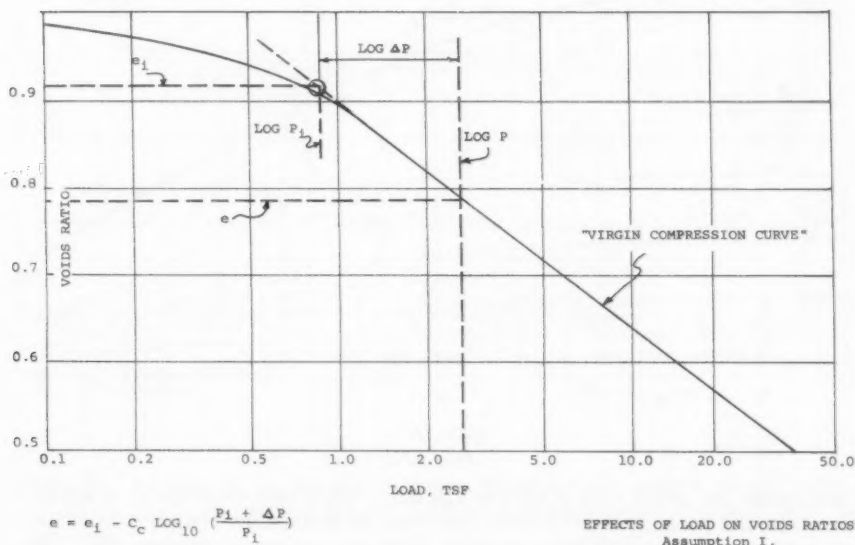


FIG. KM1

the author, will be designated here as Assumption I, and is illustrated in Fig. KM1. According to this assumption, there will be no settlement under loads less than the preconsolidation load,  $p_i$ ; and the entire curved part of the line represents recompression of a soil sample after it expanded during extraction

<sup>11</sup> "Soil Mechanics in Engineering Practice," by Karl Terzaghi and Ralph Peck, John Wiley and Sons, New York, 1949.

and subsequent submergence. This assumption is usually valid for clays overlain by sands and silts, or when the load released by excavation is quickly replaced.

There is another assumption that more truly represents conditions when the clay extends to the surface, or when excavation is extensive and the foundation is left unloaded for a long period of time, so that it expands appreciably under the release of load, and recompresses noticeably on reloading. A settlement of 1 in. occurred, for example, when backfill was placed around the raft foundation for the Shawnee Power Plant, on the Ohio River, although the soils in the foundation showed blow counts in excess of 100 blows per ft. This second assumption, here designated as Assumption II, also seems to fit many sands and silts of moderate to high density. The assumption is illustrated in Fig. KM2 for a rapidly consolidating silt. The equation given there shows some

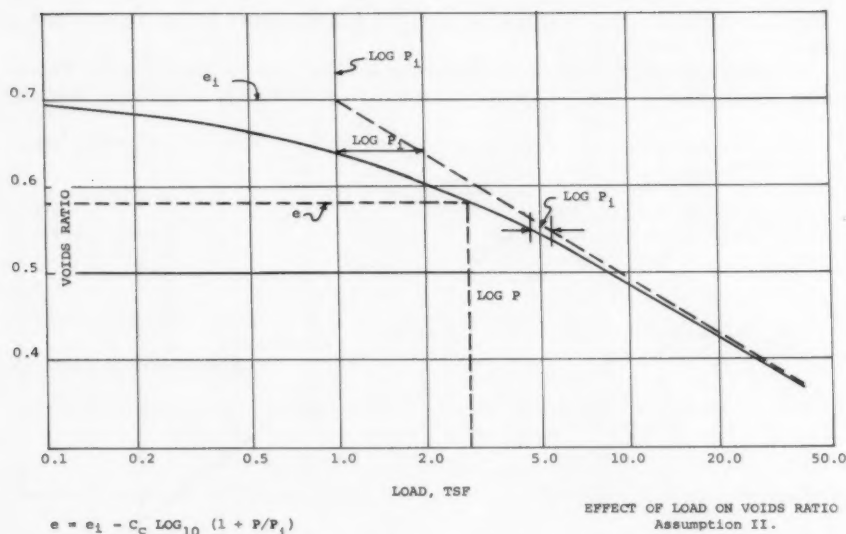


FIG. KM2

settlement for every load placed on the soil. When this assumption is applied to clays, the value  $p_i$  is practically the same as that determined by conventional methods, and the difference between the virgin compression curve and a plot of  $(p + p_i)$  versus voids ratio becomes negligible at high values of  $p$ .

Obviously, the effect of Assumption II on the author's conclusions would be to make them hold in a much higher degree. However, since the author's analysis made some simplifying assumptions which might appreciably affect these conclusions (such as taking the total settlement as a function of stress change at a depth of one third that where  $p = 0.1 p_i$ ), a modification of this analysis that is more rigorous is presented here for comparison. This modification has been applied to both Assumption I and Assumption II.

The modified analysis starts with the author's Eq. 2. The value  $\Delta e$  is taken as  $e_i - e$  from Fig. KM1. In an element of thickness  $dh$  (see Fig. KM2), the settlement is  $dS = (\Delta H/H) dh$ , and from Eq. 2,

$$dS = (e_i - e)dh / (1 + e_i) \dots \dots \dots (KM2)$$

For Assumption I, the author's Eq. 1, which is shown on Fig. KM1, is applied to Eq. KM2, giving

$$S = (1/C) \int_{h=D}^{h=h_s} \log_{10} \left( 1 + \frac{\Delta p}{p_i} \right) dh \dots \dots \dots (KM3)$$

Fig. KM3 shows the relationships between stress and depth for Assumptions I and II. In this figure, it is assumed that the preconsolidation load varies with depth and that, at the surface, there is a minimum preconsolidation load equal

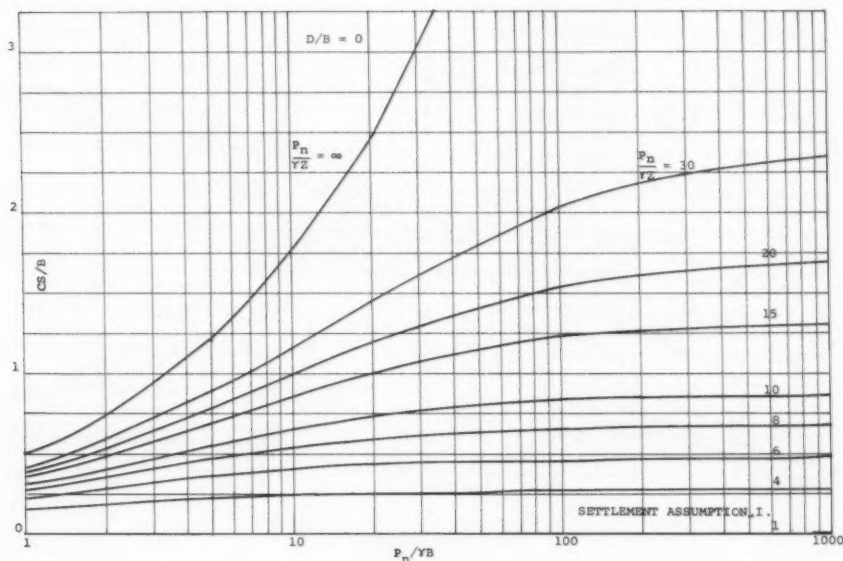


FIG. KM3

to  $\gamma Z$ . This expresses, in general terms, the assumptions made by the author.

From Fig. 3-a, holding for Assumption I, the following equations may be derived:

$$p_i = \gamma (Z + h) \dots \dots \dots (KM4)$$

$$\Delta p = \left( \frac{B}{B + h - D} \right)^2 p_n - \gamma Z \dots \dots \dots (KM5)$$

$$p_n = p_c - \gamma D \dots \dots \dots (KM6)$$

$$\frac{h_s - D}{B} = \sqrt{\frac{p_n}{\gamma Z - 1}} \dots \dots \dots (KM7)$$

Note that when  $Z = 0$  (Mr. Hough's Fig. 4-c),  $h_s = \text{infinity}$ . Practical values of  $h_s$  may be obtained by requiring that the settlement for infinite depth minus that for  $h_s$  be less than some predetermined quantity, as 0.01 ft for the most sensitive structures.

Eqs. KM4 and KM7, inclusive, have been processed by a digital computer, and the results have been plotted as curves similar to those shown by Fig. KM4. These results may be compared directly with the ones shown by the author on Fig. 5. Here we have  $D = 0$ ,  $Z = 4$  ft,  $C = 100$ ,  $S = 0.04$  ft,  $B = 6$  ft and  $p_c/\gamma$  is computed at about 38 ft. Using Fig. KM4 we have  $Z/B = 2/3$ ,  $CS/B = 0.667$ , and find the value of  $\frac{p_n}{\gamma B} = \frac{p_c}{\gamma B}$  for  $\frac{D}{B} = 0$ , at 7.5. Then  $p_c/\gamma = 45$  ft.

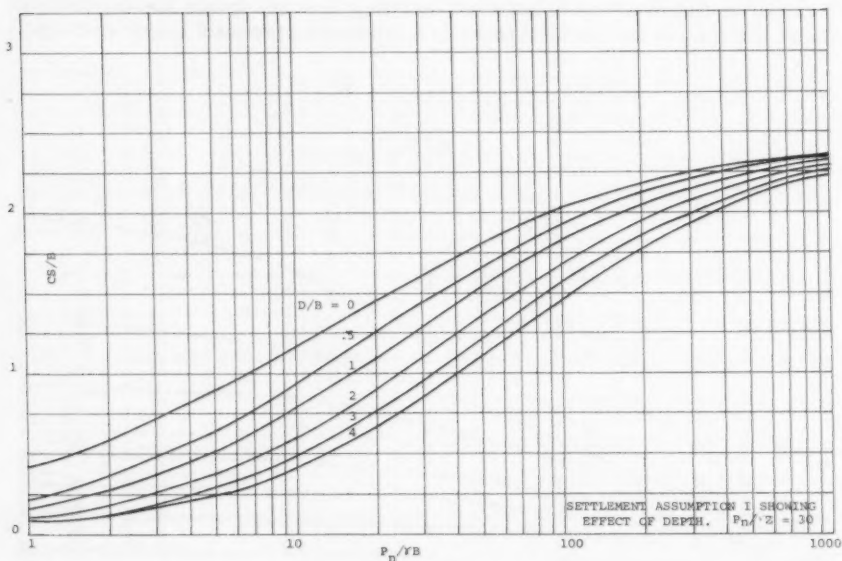


FIG. KM4

If the unit weight of soil were 100 pcf, the allowable contact pressure to limit settlement to 0.04 ft would be 4500 psf, as against 3800 psf from Fig. 5. The significant depth,  $h_s$ , would be 32 ft against less than 10 ft from Fig. 6.

Under Assumption II, we must change Eq. 1 to

$$\Delta e = C_c \log_{10} \left( 1 + \frac{p}{p_i} \right) \dots \dots \dots (\text{KM8})$$

Eq. KM3 has then been modified, giving

$$S = 1/C \int_{h=D}^{h=\infty} \log_{10} \left( 1 + \frac{p}{p_i} \right) dh \dots \dots \dots (\text{KM9})$$

Eq. KM5 is modified to

$$p = \left( 1 + \frac{h-D}{B} \right)^{-2} p_n \dots \dots \dots (\text{KM10})$$

Placing Eqs. KM5, KM10, and KM6 into KM9, computer solutions have been plotted as curves similar to those of Fig. KM5. Using the author's numerical data, we again have  $Z/B = 2/3$ ,  $CB/S = 0.667$ , and we find from the figure,  $p_c/\gamma = 18.6$  ft. For a soil having a unit weight of 100 pcf, the allowable contact pressure to limit settlement to 0.04 ft is then 1860 psf under Assumption II, as compared with 4500 psf under Assumption I. It therefore appears important to determine whether Assumption II holds, that is, whether there may be settlement at loads below the preconsolidation load. If the value of  $C$  represents an approximation of the slope of a recompression curve, it could be used with Assumption I anyway, but only up to that load at which the settlement curve approaches the virgin compression curve.

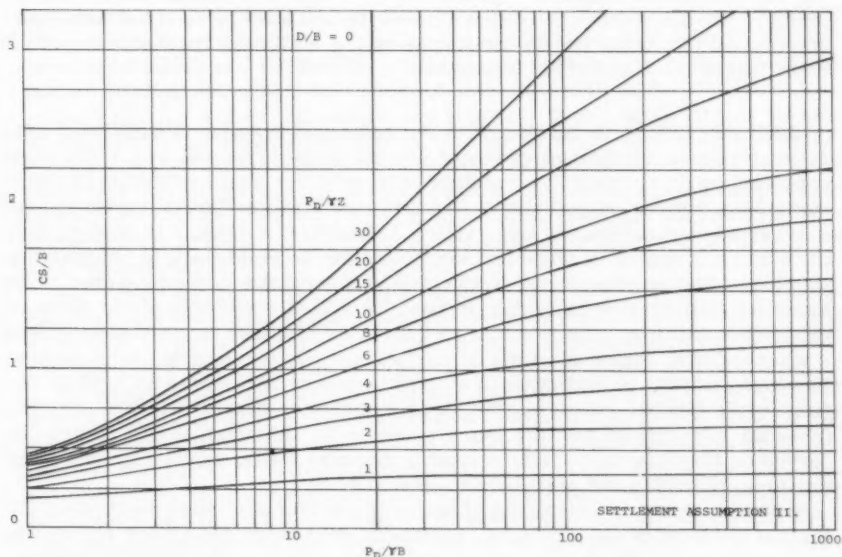


FIG. KM5

It now appears desirable to compare a commonly used method of evaluating bearing capacity with the results given herein. The simplest comparison is for a "frictionless" clay. Bearing capacity for such a soil is often determined from the equation

$$p_c = 3.7 q_u / F \quad \text{..... (KM11)}$$

where the value of  $D$  is taken at zero. Here  $F$  represents the factor of safety. Now for Great Salt Lake clays, certain Arkansas gumbos and many other soils, the preconsolidation load is approximately  $2 q_u$ . Then

$$p_c = 3.7 (\gamma Z) / (2 F) \quad \text{..... (KM12)}$$

The factor of safety is commonly taken as 3. Then  $p_c/(\gamma Z) = 0.62$  and according to Assumption I, there is no settlement, since the bearing load  $p_c$  is

less than the preconsolidation load,  $\gamma Z$ . For this same value using Assumption II, Fig. KM4 shows a maximum value of  $CS/B$  of 0.21. Values of  $C$  for clays may vary from 3 to 10, and values of  $B$  for footings (in feet) commonly run about the same. Settlements would therefore be somewhere between 0.16 and 0.7 ft, with differential settlements estimated at as much as two thirds of these values. In any case, use of a factor of safety of 3 would not keep settlement within limits that many types of modern construction can tolerate, under the stipulations of Assumption II.

It may be concluded that, under certain conditions, when it is certain that there will be no settlement for loads up to a known and appreciable value, and when this value is of the order of twice the compressive strength of a clay, the commonly used methods of evaluating the bearing capacity of the clay on the basis of stability against rupture alone will take care of settlement requirements. For other clays, as well as for sands and silts there is no such guarantee. All this holds for the conditions of Fig. KM2. As the author has indicated, there are many other conditions.

LEV ZETLIN,<sup>12</sup> M. ASCE.—With the increased practice of employing ultimate theory in reinforced concrete design, with increased use of flat plate construction in multi-story buildings, and with the rapid evolution of space structures, foundation design based on settlement rather than on rupture of the soil becomes of paramount importance. Structural members in modern construction are becoming more and more shallow in comparison to construction practice a decade or so ago. This trend is natural due to the progress in the theory of structural engineering and our increased knowledge of the material properties, and it results in a more rational use of construction materials and a more uniform factor of safety throughout various portions of the structure. However, because of shallower structural members, structures as a whole become more flexible, affecting their load distribution characteristics. As differential settlements become more pronounced, detrimental effects are observed on finishing materials (enclosing masonry walls, doors, windows, floor finishing, plumbing and mechanical installations).

Although less economical and of non-uniform factor of safety throughout the various portions of the structure, relatively deep beam and girder framing is more apt to redistribute load due to differential settlements than the modern, more flexible structures with shallow members.

When differential settlement occurs in more rigid buildings, load is relieved from the more settled columns to the less settled columns. With more flexible modern structures, although load distribution takes place to some degree in the floor slabs, affecting safety of the structure favorably, load distribution among various columns in the building is not as pronounced.

It is with respect to these more flexible structures that Mr. Hough's paper makes a distinctive contribution to structural engineering—although the importance of differential settlements could not be overlooked in the design of any other structures.

There is no doubt that foundation design as commonly practiced, particularly in architectural type buildings, did not keep pace with the progress of the design of the superstructure. This applies not only to safety, cracking, etc., of a structure due to foundation causes, but also to the economy of foundation

<sup>12</sup> Cons. Engr., New York, N.Y. Visiting Professor of Civ. Engrg., Manhattan College, New York, N.Y.

construction. Arbitrary bearing pressures provided by the building codes, or allowable bearing pressures as determined by standard penetration tests and correlated with the well-known ultimate bearing capacity theories, vary within a wide range for the same type of soil. Because of these discrepancies, it is not quite uncommon in the engineering profession to be overly conservative (erroneously) and to use a very low design bearing pressure and, thus, increase the cost of foundations. In order to preclude differential settlements, pile foundations have been used in soils adequate for spread footings or for other shallow foundations. In cases like these, economy could have resulted if differential settlements of spread footings could have been reliably predicted.

The writer has studied<sup>13</sup> the correlation between various ultimate bearing capacity formulae (such as Eq. 38, Meyerhoff's and others) and the allowable pressures as prescribed by building codes, with a great number of footings both in cohesionless and cohesive soils, in buildings affected by the foundations (cracks in walls, beams, etc.). Although the allowable pressures by ultimate bearing formulae and the building codes for footings in a specific type of soil varied within an extremely wide range, it was obvious from the study that damage to buildings was not related to soil failure. It was apparent that damage was more related to size and depth of footings as well as to the type of structural framing in the buildings. In other words, differential settlements, which were the cause of the damage, bore no relationship to bearing pressures based on ultimate bearing capacity formulas or building codes, no matter what value of the bearing pressure from within the wide range of allowable pressures had been considered.

Mr. Hough's paper is presented in an extremely lucid manner. The numerous available curves offer the possibility of their direct application to design of footings. The highlight of the paper is the correlation between the bearing capacity index and the stress distribution in the soil mass under a footing.

Although not detracting from the value of the paper, the writer is somewhat puzzled by Fig. 6, which shows correlation between  $C$  and  $H$ . The depth of significant stress is obviously of interest to the structural engineer in specifying depth of borings in subsurface investigation.

In accordance with the definition of  $H$ , the depth depends only on the characteristic of the footing ( $D$ ,  $B$ , Contact Pressure  $p_c$ ), and  $60^\circ$  approximation in one dimensional stress distribution, but not on the soil properties (except  $\gamma$  which is an independent variable). Namely, the  $C$  value of the soil (and, therefore,  $S$ ) does not affect the depth of significant stress. In Fig. 6, both  $C$  and  $S$  have been correlated with  $H$ . From a purely mathematical viewpoint, it is possible to have an imaginary relationship between one function (such as  $H$ ) with an outside parameter (such as  $C$ ) which does not affect the function, provided the independent parameter is damped by another parameter (such as  $S$ ) which in turn is related to the first independent parameter. In this respect, it is possible to plot a curve which does represent an apparent relationship between the function  $H$  and outside parameters  $C$  and  $S$ . The variation of such a curve, however, would not depend on the independent parameters. Fig. 6, on the other hand, does show a definite relationship in variation between  $H$ , and  $C$  and  $S$ .

Although this reasoning is purely mathematical, there is some confusion from a purely practical viewpoint. Considering a specific type of soil with a fixed value of  $C$ ,  $H$ , according to Fig. 6, increases with the increase in the

<sup>13</sup> "Research Report on Foundation Design," by Lev Zetlin, Cornell Univ., 1952.

permissible  $S$ . Since  $S$  represents the change in thickness of the soil layer within which the stresses are considered, it follows that the actual settlement  $S$  would increase with the increase in the thickness of this soil layer  $H$ . Hence, it seems reasonable to assume that as the permissible settlement gets smaller (and, therefore, should be computed more accurately), a larger soil layer affected by vertical stresses should be considered in predicting settlement. In other words, the depth of significant stress should be inversely proportional to permissible settlement.

The writer realizes that, in accordance with its definition in Eq. 8, the depth of significant stress is not dependent on the settlement (actual or permissible). However, since Fig. 6 correlates depth of significant stress to the permissible settlement, this point is brought up for Mr. Hough's further clarification of the curves in Fig. 6.

This paper offers a new tool for a rational design and proportioning of footings. In buildings with basements of varying depths and, hence, of varying footing elevations, it enables the engineer to design footings for uniform settlement. On the other hand, it is conceivable that by varying elevations of footings for uniform settlement, more economical foundation design could be attained. It would be most encouraging and helpful to the engineering profession to see further development of the subject matter treated in the paper, such as extensive investigation of the relationship between the bearing capacity index and the standard penetration test, effects of overlapping of stresses under adjacent footings, etc. In the opinion of the writer, design of footings based on settlement, and checked for failure of the soil will, eventually become a routine practice in building design. Mr. Hough's paper is a milestone in this direction.

BRAMLETTE McCLELLAND,<sup>14</sup> M. ASCE.—The author has brought new emphasis to the need for proportioning footings, not only to provide safety against rupture but also to avoid damaging settlement due to soil compression. This need is traditionally stressed in academic courses in soil mechanics and is recognized in practice by most consulting soils engineers. In spite of this, it is probably true that a substantial number of today's new structures—perhaps far more than a majority—have foundations designed solely on the basis of soil strength. To this extent, the writer finds a substantial area of agreement with the author and it is hoped that his paper will stimulate greater attention to soil compression as a criterion for selecting footing sizes. However, securing this added attention requires something other than improving our techniques of analysis.

One principal cause of the neglect of soil compression in footing design, and the corresponding over-emphasis on soil strength, may be poor communication between soils engineers and structural designers. Quite frequently the soil conditions at a site are investigated and analyzed by persons or organizations other than (and sometimes remote from) those who are finally responsible for designing the footings to be used at that site. Since competent soils engineers are well familiar with analytical procedures used to investigate both strength and compressibility, the fact that one of the criteria is frequently neglected suggests that the soils engineers fail to make their knowledge known to or useable by the structural designer.

Many structural engineers, and in particular the structural squads of large organizations, prefer to work with a single "soil bearing value" in preparing

<sup>14</sup> Pres., McClelland Engrs., Inc., Houston, Tex.

plans for a project. This preference is partly a carry over from the past and is partly a misguided desire for simplicity and efficiency. The structural designer, therefore, cannot be counted on to "pull" information on settlement from the soils engineer. Also, if settlement criteria are presented to him in either vague, inconclusive, or complicated form, he may well ignore them or elect to use deep foundations to evade the problem.

What is needed, then, is a lucid means of informing the structural designer how the factors within his control relate to footing settlement at a particular site. One such method employs a diagram in the form of Fig. 10(b), which is deserving of additional attention. Such a diagram consists of a family of curves of equal settlement, plotted on a graph of column load versus footing size. An example of this type of presentation, for a site in the Gulf Coastal Plain, is given in Fig. M1. The independent variables in this diagram are those best

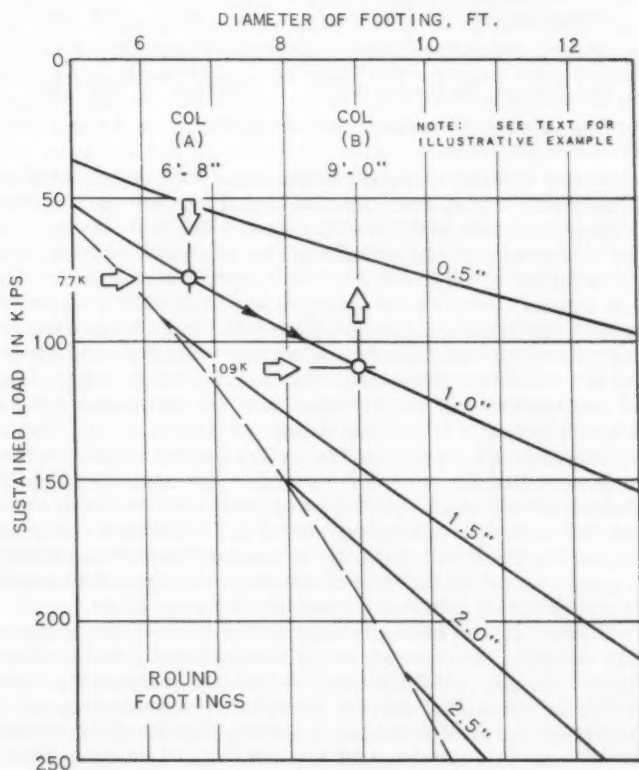


FIG. M1.—FOOTING SETTLEMENT CURVES

known to, or in the control of, the structural designer; the dependent variable, settlement, is one on which he must pass judgment as to permissible magnitude. All other variables, such as compression ratio, ground water level, density and depth of significant stress, are expressed in the shape of the settlement curves and are of no concern to the structural designer. Combinations of footing size and column load, falling to the left of the dashed line in Fig. M1, result

in an inadequate factor of safety with respect to soil rupture, indicating that soil rupture must not be ignored in this particular case.

An important consideration in proportioning footings on cohesive soils, using data of the type given in Fig. M1, was not dealt with by Mr. Hough. Column loads for this purpose should be sustained loads rather than total loads. On the other hand, total dead plus live loads are to be considered when determining or checking the footing size with respect to safety against rupture. As an illustrative example, the curves on Fig. M1 were used to select sizes of adjacent footings carrying total column loads of 105 and 149 kips and sustained loads of 77 and 109 kips, respectively. Results of this computation are:

Item	Col. A	Col. B
Total Column Load	105 kips	149 kips
Sustained Column Load	77 kips	109 kips
Footing Diameter	6 ft 8 in	9 ft 0 in
Net Bearing Pressure (total)	3000 psf	2340 psf
Computed Settlement	1.0 in	1.0 in

The steps followed in these calculations are illustrated in part on Fig. M1, and conform to the author's suggestion that the footing size for the smallest column load be selected first on a basis of safety against rupture. In this case, bearing capacity computations had established an allowable bearing pressure of 3000 psf for a factor of safety of 2.0 with respect to soil shear. Using the smaller total column load in the foregoing table and the allowable bearing pressure just stated, the footing diameter of 6 ft 8 in. was selected for column A. Entering the chart with this footing size and the sustained load for column A, a settlement of 1.0 in. was determined. Next, to select a footing size for column B, the chart was entered with the sustained load for that column. For an equal settlement with column A, it was found that the proper footing diameter was 9 ft - 0 in. Any footing size selected by such a procedure should be checked, however, to be sure that the pressure created by the total dead plus live loads does not exceed the allowable bearing pressure. In this case, the total net pressure for the column B footing was found to be 2340 psf. At sites where there is exceptional horizontal continuity in soil compressibility, it may be permissible to permit some inequality of settlement in applying the foregoing procedure, thereby effecting significant economy in footing sizes.

Settlement data can be readily communicated in the form of Fig. M1 and can be easily used as a criterion for proportioning footing sizes. This success does not depend, however, on which computational or analytical procedure was used to develop the settlement curves, provided reasonable accuracy was obtained. This writer has serious doubts, however, that the author's procedures can be used with adequate accuracy for any more than a few exceptional cases. Inaccuracies will come from forcing the many variables usually dealt with to fit into the inflexible framework of equations such as 9, 10 and 14, also from assuming an average  $C_c$  to apply to the full depth of significant stress, sometimes even from assuming that  $C_c$  is a constant at all. Direct or "long-hand" computations of settlement for sufficient combinations of column load and footing size, to determine a design chart such as Fig. M1, are not so burdensome as to be impractical.

Finally, it is feared that dependence on design charts such as Fig. 5 and Fig. 6, and use of abstract concepts such as the author's "allowable pressure index," would keep the designer too remote from the physical problem itself and interfere with his use of judgment, an indispensable quality in foundation analysis. For example, Fig. 6 may adequately represent the mathematical relationships of Eqs. 31 and 32; yet, its literal interpretation suggests that "significant stress" may be found at 80-ft depth beneath an 8-ft footing under some conditions. It is certain that such misuse of the procedure was not intended by Mr. Hough, but the danger of such errors in the hands of the uninitiated appears to be inherent.

JAMES CHINN,<sup>15</sup> A. M. ASCE.—Mr. Hough has presented an excellent paper which could well revolutionize the procedures used for proportioning footings.

Unlike most authors, Mr. Hough has considered the mechanics of settlement to be basically the same for all types of soils, that is, he considers the settlement which is of significance in foundation design to be one dimensional consolidation with all soils exhibiting a straight line  $e$  versus  $\log p$  relationship.

It is generally agreed that settlement in clays can be adequately predicted by the one-dimensional consolidation theory, but those few authors who mention settlement in sands refer to stress-strain curves from triaxial tests with constant lateral pressures. The specimens bulge in such tests, and strain is not one dimensional. It is not likely that the consolidation which takes place under a footing is absolutely one dimensional, but it seems reasonable to the writer that any lateral strains would be small. It does not seem reasonable that the lateral pressures in the soil below the edges of footings remain constant as in the triaxial tests, but they would be expected to increase.

The question of whether settlement is one-dimensional consolidation or not could be purely academic, however. It takes little contemplation to discover that we use daily procedures based on assumptions which are not absolutely true: If the author's theory yields results correct to the necessary accuracy, it matters not that his assumptions might be slightly in error. It is hoped that soils engineers will try the procedures proposed and observe their accuracy in practice.

The amount of error involved in approximating the average unit change in thickness by that occurring at  $h_g/3$  could probably stand investigation, as obviously, it represents different relative values for the three cases considered; surface footing, open excavation, and surcharged footing.

Keying Figs. 3 and 9 to standard penetration resistance is highly desirable as a simple, practical means for determining the bearing capacity index and presumptive bearing values or soil rupture strength. Unfortunately, however, the "standard" penetration test is far from standard. The author's  $N$ -values agree with those listed in soils engineering texts, but considerable variation has been reported,<sup>16</sup> and is recognized by soils engineers who have encountered different techniques used in performing the penetration test. Full details of performing the tests which yielded the values shown in soils engineering texts should be made generally available, such as weight of drill rod, method of lifting, dropping, and guiding hammer, etc. This same procedure should be used by all soil testing firms, or results of other procedures should be correlated.

<sup>15</sup> Assoc. Prof., Univ. of Colorado, Boulder, Colo.

<sup>16</sup> "Relative Density and Shear Strength of Sands," by T. H. Wu, *Proceedings, ASCE*, Vol. 83, No. SM 1, January 1957, pp. 1161-9.

The uniform building code of the Pacific Coast Building Officials Conference, like the 1944 Boston Building Code cited by the author, adjusts allowable bearing pressures for depth but, in addition, adjusts them for width of footing. Mr. Hough's method appears to do everything this Code does and more. Further, it appears that his method makes more sense than the Code. It is, however, subject to some of the same limitations. It is not applicable without modification to a soil which is layered within the depth of significant stress nor to a situation where the stress increment under a footing is affected by adjacent footings. The modifications for these cases do not appear complicated, however.

Additional factors worthy of consideration are:

1. Clays may have been preloaded, which fact should be recognized in the value of  $p_i$  used. This can be handled as the open-excavation case.
2. The load which produces settlement may not be the full design dead plus live load required by the building code. This might be dead load plus maximum expected (rather than code specified) live load for sands and dead load plus any live load always on the structure for clays. Design against soil rupture, however, should probably be for the full code requirement.

Use of presumptive bearing values as the criteria for safety against soil rupture is not too satisfactory, in the writer's opinion. These values, as the author has stated elsewhere<sup>17</sup> consider both excessive settlement and soil rupture but do not indicate which controlled in setting the allowable value. Since no modification is made for location of water table, the values might be unconservative in guarding against soil rupture under narrow footings on sand with a high water table. In cases where settlement controlled, the values are probably far too conservative.

Most building codes have a statement to the effect that if bearing capacity of soil is not determined by adequate tests, contact pressures shall not exceed those given in the code's table of presumptive bearing values. The writer feels that the philosophy behind presumptive values is that these are minimum values which can be used (with the exception of the previous paragraph) only when visual or very crude tests are run on the soil. Where more complete tests are run, and proper foundation design procedures are used, higher values should be allowed.

Rather than using presumptive bearing values to guard against soil rupture, it is felt that ultimate bearing capacities (such as those in Fig. 9) divided by appropriate factors of safety should be used.

Finally, the writer feels that footings should be designed for a tolerable differential settlement and checked for soil rupture if deemed necessary. The tolerable differential settlement is dependent upon the type of structure but is at least 3/4 in. for structures of most any building material. The absolute settlement can be arrived at as experience is gained in the accuracy of settlement prediction. As a starting point, since settlement represents only an order of magnitude rather than an exact figure, it seems reasonable that differential settlements will generally not exceed 50% of the predicted settlement. To limit differential settlement to 3/4 in. therefore, footings would be designed for absolute settlement of 1-1/2 in. If experience shows that 50% is too high, this figure can be reduced and higher absolute settlements used.

Mr. Hough is to be commended for presenting the first new thoughts on footing design in over a decade. The writer feels that too many "facts" and rules

<sup>17</sup> "Basic Soils Engineering," By B. K. Hough, Ronald Press, New York, 1957, p. 277.

have been accepted in soils engineering without adequate proof, and it is hoped that this paper will set engineers to thinking of other procedures which should be questioned.

*Phragmites australis* (Cav.) Trin. ex Steud.

11. The following table shows the number of people who attended the concert in each age group.

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A STATISTICAL STUDY OF SOIL SAMPLING<sup>a</sup>

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Discussion by John A. Focht, Jr.

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JOHN A. FOCHT, Jr.,<sup>1</sup> M. ASCE.—The statistical studies reported in this paper supply a needed guide as to the number of tests necessary to obtain an average satisfactory value for the index properties of a given soil. The implication of the paper is that an average of results from single tests on five different samples of a particular soil type, will differ only slightly from the true average at least 95% of the time. The writer poses this question to the authors—"Are the differences in the test results for a particular soil horizon due to normal variation in soil properties or to variations inherent in the test technique?"

Results of studies to determine the uniformity of (or variation in) results of liquid limit tests using "uniform" materials were reported by Raymond F. Dawson<sup>2</sup> in 1959. In some of the test series performed in that study, deliberate efforts were made to eliminate, or at least to reduce to a minimum, variations between individual test specimens. Nevertheless, the range of scatter of liquid limit values from a number of tests amounted to about 10% of the average of all the results. Expressed in the same fashion as the table near the end of the authors' summary, any individual value was generally  $\pm 5\%$  of the average of a group of about 10 to 20 tests. This plus-or-minus range is almost identical with that stated in the summary. The implication of this comparison is, then, that scatter of the liquid limit test results, due variations in test technique, might have produced a large part of the random scatter found by the authors between tests on samples from the same horizon. If technique variation is significant in the liquid limit test, it could also be present in the other two index tests studied by the authors.

Regardless of the source of the variations, it is extremely important that engineers, using values of single index tests or even average values from five tests, recognize that variations of some magnitude do develop. To this end, as well as indicating the possible range of variation, this paper will be extremely valuable.

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<sup>a</sup> October, 1959, by Thomas H. Thornburn and Wesley R. Larsen.

<sup>1</sup> Chf. Design Engr., McClelland Engrs., Inc., Houston, Tex.

<sup>2</sup> "Investigations of the Liquid Limit Test on Soils," by Raymond F. Dawson, presented to the Annual Meeting, ASTM, June, 1959.

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CONSTRUCTION PORE PRESSURES IN AN EARTH DAM<sup>a</sup>

Discussions by P. J. Moore and Gideon Yachin

P. J. MOORE,<sup>1</sup> A. M. ASCE.—The test apparatus for the measurement of pore pressures and embankment settlement, which has been described by Mr. Li, is similar to that apparatus which was installed in Adaminaby Dam, Australia. This dam is an earth and rockfill structure, 386 ft high and was constructed by the New South Wales Department of Public Works, for the Snowy Mountains Hydro Electric Authority. The test apparatus included 44 embankment-type and 14 foundation-type piezometer tips, and three crossarm installations for the measurement of embankment settlement.

As in the case of Quebradona dam, the placement moisture content was lower than the anticipated moisture content for which the dam was designed. The design of Adaminaby was based on a placement moisture content equal to the optimum of 40-blow Proctor compaction in a 1/20 cu ft mould. The actual placement moisture content was 10.8% compared with the average optimum moisture content of 11.6%. This rather small variation in placement moisture resulted in a very large decrease in construction pore pressure. For optimum placement large construction pore pressures were anticipated; however, the construction pore pressures which were measured by the piezometers were negligible. The period of construction of the embankment was from January, 1956, to May, 1958.

At Adaminaby Dam, a variation was, also, obtained between the field compression curves at different levels in the dam. Referring to Fig. M1, it is seen that the lower strata (numbering of crossarms is from the bottom upwards) is considerably more compressible than the upper strata. In this case, however, the difference appears to be caused by variations in placement moisture, as indicated in Table 1.

However, since Mr. Li states that the soil type and placement conditions in Quebradona dam are practically unchanged throughout the full height of the settlement installation, it appears unlikely that the differences in field compression curves would have the same cause as in the case of Adaminaby Dam. Examination of Figs. 8 and 9 indicates that the measured construction pore pressures are less than hydrostatic pressure, if, as is suggested by the plot of water level in the embankment at the settlement installation in Fig. 7, the water table in the embankment remained close to the elevation of the fill during the period of construction. If this is so, then the presence of downward seepage pressures is suggested. These seepage pressures would increase the effective stress (to a greater extent on the lower strata than on the upper) over that calculated on the basis of the pore water being stationary. This increase in effective stress would tend to bring the field compression curves in Fig. 6 closer

<sup>a</sup> October, 1959, by C. Y. Li.

<sup>1</sup> Superv. Engr., Dept. of Pub. Wks., N.S.W., Australia.

together. Would the author have any further information which would indicate the position of the water table in the embankment or which may suggest the presence of downward seepage through the embankment?

The writer would appreciate Mr. Li's comments regarding the benefit to be gained in design by the use of a more refined method of calculation of pore

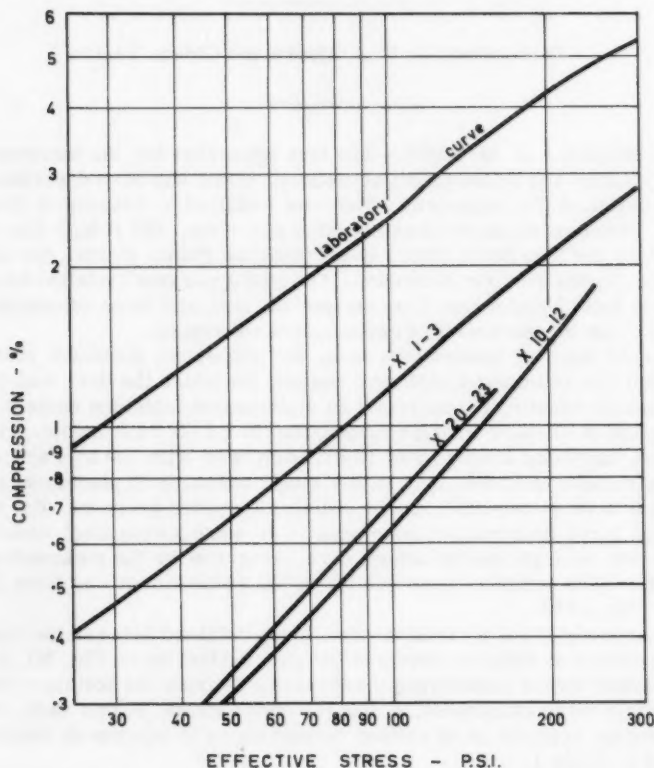


FIG. M1.—COMPRESSION CURVES FOR ADMINABY DAM

pressures based on laboratory consolidation tests, in view of the following observations:

(a) As seen in Fig. M1, the average laboratory compression curve (derived from consolidation tests on representative embankment material) for Adaminaby Dam, is quite different from the field compression curves. Any calculation of anticipated construction pore pressure which is based on this laboratory curve would yield values of pore pressure higher than would be measured in the embankment.

(b) In both Quebradona and Adaminaby Dams the placement moisture content was lower than that anticipated. In the latter case, it has been shown that this decrease in placement moisture resulted in a very large decrease in construction pore pressures.

TABLE 1.—AVERAGE PLACEMENT CONDITIONS BETWEEN CROSSARMS

Crossarms	Placement Moisture, in %	Optimum Moisture, in %	Placement Dry Density, in pounds per cubic foot	Maximum Dry Density, in pounds per cubic foot
1 - 3	11.5	12.1	116.1	120.1
10 - 12	10.7	11.8	117.7	120.3
20 - 23	10.3	11.5	116.4	121.4

GIDEON YACHIN,<sup>2</sup> M. ASCE.—The author is to be commended for his fine presentation of the pore pressure and settlement data recorded during and after construction of Quebradona Dam. However, the interesting attempt to try and predict embankment, pore pressure buildup, requires careful consideration.

Mr. Li, in his analysis, applied 5 psi effective stress increments and obtained pore pressure versus compression curves for 1/5, 1/4, 1/3, and 1/2 pore pressure dissipation after the application of each effective stress increment. The author admits that the shortcoming of his method lies in assuming the percentage of dissipation, based on judgment. However, it appears that by using increments of effective stress other than 5 psi, with the same degree of pore pressure dissipation as used by the author, the resulting curves are completely different from the ones presented by Mr. Li. A comparison between 5 psi and 2.5 psi effective stress increments, assuming 1/3 pore pressure dissipation after each increment, is presented in Fig. Y1. Therefore, the writer assumes that Mr. Li combined both the magnitude of the stress increment and the degree of dissipation in his reference to engineering judgment. However, if the writer's assumption is correct, then the attempt made by the author to connect the points representing the pore pressure dissipation at each stage is misleading.

The correct curve for the assumed conditions is the original zigzagged curve represented by 1a-1b-2a-2b, etc., in Fig. Y1. Let us assume that for the previously postulated conditions it is required to estimate the pore pressure when a total of 10 psi effective stress is attained in the embankment. The 10-psi line intersects the  $\bar{\sigma}$  versus  $\Delta$  curve at  $\Delta = 2.9\%$ . Pore pressure is 17.5 psi and 9 psi for no dissipation and 1/3 dissipation, respectively. However, the author's suggested curve indicates 6.5 psi for the above conditions. Curve 1a-1b-2a-2b, etc., as presented in Fig. Y1, differs beyond point 2a from the author's original curve. It is possible that by substituting  $\bar{V}_{a2} + H \bar{V}_{w2}$  in Eq. 1, the author plotted the pore pressure from the obtained solution which actually yields the value of  $U_{2a}-U_{1b}$ . As the above mentioned statement is a pure speculation on the part of the writer, computations of the first two stages are presented for the sake of comparison:

<sup>2</sup> Civ. Engr., Gannett Fleming Corddry and Carpenter, Inc., Harrisburg, Pa.

Initial Conditions:

$$\gamma_{\text{dry}} = 96.2 \text{ pcf}; \quad w = 25.4\%; \quad G = 2.71$$

$$e = \frac{G \gamma_w - \gamma_d}{\gamma_d} = \frac{2.71 \times 62.4 - 96.2}{96.2} = 0.7578$$

$$n = \frac{e_o}{1 + e_o} = \frac{0.7578}{1.7578} = 0.4311; \quad S_o = \frac{G w}{e_o} = \frac{2.71 \times 25.4}{0.7578} = 90.83\%$$

Stage I.—Assume a load was placed to yield an effective stress of 5 psi. From  $\bar{\sigma}$  versus  $\Delta$  curve, this will cause a  $\Delta = 1.7\%$  under no drainage conditions.

$$U_{1a} = \frac{P_a \Delta}{\bar{V}_o + H \bar{V}_{w-\Delta}} \dots\dots\dots (Y1)$$

For initial conditions assume  $P_a = 11.2$  psi = Atmospheric pressure at 6,700 ft above mean sea level:

$$\bar{V}_{vo} = 100n = 43.11\%$$

$$\bar{V}_{wo} = (0.9083)(43.11) = 39.16\%$$

$$\bar{V}_{ao} = \bar{V}_{vo} - \bar{V}_{wo} = 43.11 - 39.16 = 3.95\%$$

Substituting in Eq. Y1

$$U_{1a} = \frac{11.2 \times 1.7}{3.95 + 0.0198 \times 39.16 - 1.7} = 6.29 \text{ psi}$$

Allowing  $1/3$   $U_{1a}$  dissipation

$$\bar{\sigma}_{1b} = \bar{\sigma}_{1a} + \frac{1}{3} U_{1a} = 5 + \frac{6.29}{3} = 7.1 \text{ psi}$$

The increased value of  $\bar{\sigma}$  due to dissipation of pore pressure causes additional volume change of 0.5% as indicated by  $\bar{\sigma}$  versus  $\Delta$  curve, Fig. Y1.

By the author's assumption  $S_{1a} = S_{1b}$

$$\Delta e = \frac{\Delta(1 + e_o)}{100} = \frac{1.7 \times 1.7578}{100} = 0.0299$$

$$\text{therefore} \quad e_{1a} = e_o - \Delta e = 0.7578 - 0.0299 = 0.7279$$

$$S_{1a} = S_{1b} = \frac{w G}{e_{1a}} = \frac{25.4 \times 2.71}{0.7279} = 94.56\%$$

$$\text{After dissipation} \quad \Delta = 1.7 + 0.5 = 2.2\%$$

$$\Delta e = \frac{2.2 \times 1.7578}{100} = 0.0387$$

$$e_{1b} = e_o - \Delta e = 0.7578 - 0.0387 = 0.7191$$

$$S_{1a} = S_{1b} \quad \text{therefore, there is a reduction in water content}$$

$$W_{1b} = \frac{S_{1b} e_{1b}}{G} = \frac{94.56 \times 0.7191}{2.71} = 25.09\%$$

Stage II.—Initial conditions at 1b:

$$\text{Absolute pore pressure} = p_a + \frac{2}{3} U_{1b} = 11.2 + 4.2 = 15.4 \text{ psi}$$

$$\bar{V}_{v1b} = n_{1b} = \bar{V}_{v0} - \Delta = 43.11 - 2.2 = 40.91\%$$

$$\bar{V}_{w1b} = \frac{\bar{V}_{v1b} 1b}{100} = 0.9456 \times 40.91 = 38.68\%$$

$$\bar{V}_{a1b} = V_{v1b} - V_{w1b} = 40.91 - 38.68 = 2.23\%$$

Another load increment to yield an additional 5 psi effective stress is applied:

$$\bar{\sigma}_{2a} = \bar{\sigma}_{1b} + 5 = 7.1 + 5 = 12.1 \text{ psi}$$

From  $\bar{\sigma}$  versus  $\Delta$  curve,  $\Delta_{2a} = 3.4\%$ , or an additional compression of  $3.4 - 2.2 = 1.2\%$ .

Substituting in Eq. Y1

$$U_{2a} - U_{1b} = \frac{15.4 \times 1.2}{2.23 + 0.0198 \times 38.68 - 1.2} = 10.29 \text{ psi}$$

therefore

$$U_{2a} = U_{1b} + 10.29 = 4.2 + 10.29 = 14.49 \text{ psi}$$

Assuming  $1/3 U_{2a}$  dissipation = 4.83 psi,  $\bar{\sigma}_{2a}$  will be increased by the same amount:

$$\bar{\sigma}_{2a} = 12.1 + 4.83 = 16.93 \text{ psi}$$

From  $\bar{\sigma}$  versus  $\Delta$  curve  $\Delta_{2b} = 3.85\%$  or an additional compression of  $3.85 - 3.4 = 0.45\%$

$$\Delta e = \frac{A_{2a}}{100} (1 + e_0) = \frac{3.4 \times 1.7578}{100} = 0.0598$$

After dissipation

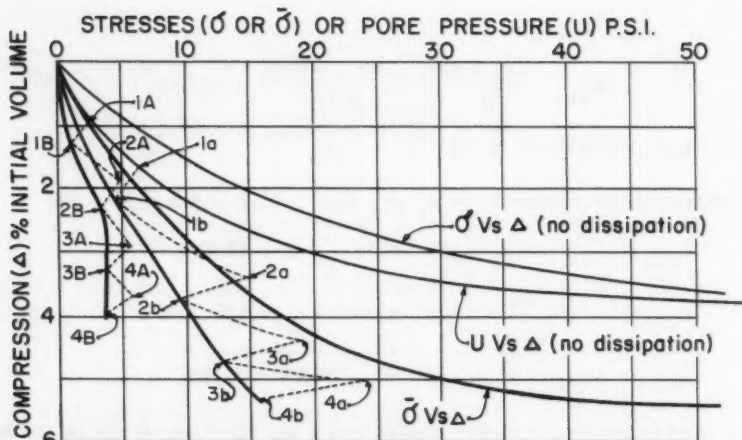
$$e = e_0 - \Delta e = 0.7578 - 0.0598 = 0.6980$$

$$S_{2a} = S_{2b} = \frac{W_{1b} G}{e_{2a}} = \frac{25.09 \times 2.71}{0.6980} = 97.41\%$$

$$\Delta e = \frac{\Delta_{2b}(1 + e_0)}{100} = \frac{3.85 \times 1.7578}{100} = 0.0677$$

$$e_{2b} = e_0 - \Delta e = 0.7578 - 0.0677 = 0.6901$$

$$w = \frac{S_{2b} \times e_{2b}}{G} = \frac{97.41 \times 0.6901}{2.71} = 24.81\%$$



The zigzagged line 1a,1b,2a,2b,3a,3b,4a,4b represents pore pressure (U) Vs  $\Delta$  for 5 psi effective stress increments and 1/3 pore pressure dissipation after each increment.

The zigzagged line 1A,1B,2A,2B,3A,3B,4A,4B represents pore pressure (V) Vs  $\Delta$  for 2.5 psi effective stress increments and 1/3 pore pressure dissipation after each increment.

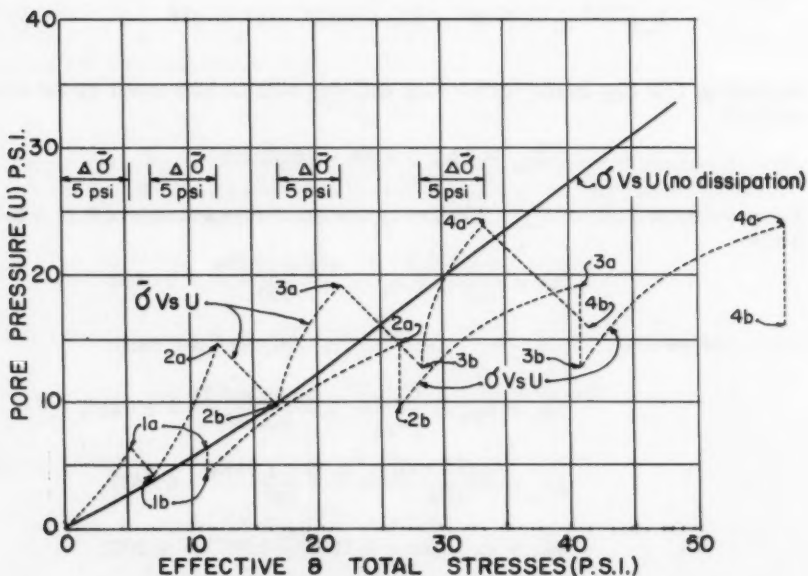


FIG. Y1.—PORE PRESSURE, COMPRESSION AND STRESS RELATIONSHIP

The prediction of pore pressure buildup in embankment is a function of the rate of construction. Any attempt to predict pore pressure dissipation should include the assumed or suggested construction schedule.

The theory of consolidation of saturated soil may be applied to predict pore pressure dissipation in partially saturated soils. Karl Terzaghi<sup>3</sup> suggested that the volume of water which drains out of clay with entrapped gas bubbles is greater than the decrease of voids of the clay compared to similar conditions in completely saturated clays. This statement is based on the assumption that gas, or air, being compressible, may change volume even under conditions of no drainage. A. W. Bishop,<sup>4</sup> also quoted by the author, admits that for soils containing relatively little free air the air bubbles will find difficulty in moving. Bishop concludes that the assumption of air and water mixture leaving a compressed soil sample at the same proportion by weight as that remaining in it, is a conservative one. In most embankments, the degree of saturation at Proctor optimum is over 80%. In the case of Quebradona Dam the initial degree of saturation at placement is 90.83%.

The following assumptions are made in the proposed analysis:

a. The ultimate degree of soil compression does not depend on the degree of saturation, after its placement characteristics have been established. Therefore, a remolded compacted sample will undergo an identical ultimate volume change, even if its degree of saturation will be altered after initial compaction.

b. Pore pressure dissipation in soils with an appreciably high degree of saturation is achieved by process of drainage of the incompressible fluid only; that is, water.

c. The compression of air is instantaneous, thus, independent of time factor.

The consolidation characteristics of two identical soil samples that differ only in their degree of saturation is presented in Fig. Y2. From Fig. Y2 it may be deduced that for the change of volume of  $e_1 - e_2$  the volume of escaped water will be  $e_1 - e_2$  and  $e_1 - e_2 + (e_0 - e_1)$  for saturated and nonsaturated samples, respectively.

Time-consolidation relationship is expressed as follows:

$$T = \frac{c_v t}{H^2} \dots\dots\dots (Y2)$$

where  $T_v$  is the time factor,  $c_v$  is the coefficient of consolidation,  $t$  is the time of consolidation, and  $H$  is the length of the drainage path.

From the theory of consolidation

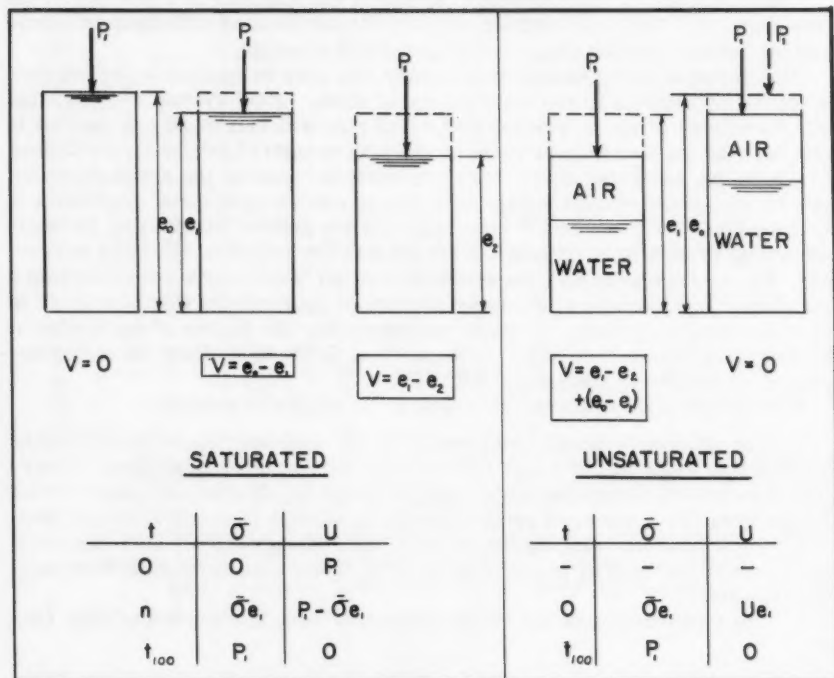
$$c_v = \frac{k}{\gamma_w m_v} \dots\dots\dots (Y3)$$

and

$$m_v = \frac{a_v}{1 + e_0} \dots\dots\dots (Y4)$$

<sup>3</sup> "Effect of Gas Bubbles on the Rate of Drainage of a Bed of Ideal Clay thru Its Base," by Karl Terzaghi, *Theoretical Soil Mechanics*, 1943.

<sup>4</sup> "Some Factors Controlling the Pore Pressure Set Up during the Construction of Earth Dams," by A. W. Bishop, *Proceedings, 4th International Conference on Soil Mechanics and Foundations, Volume II*, 1957.



### LEGEND

- $e_0$  = Initial void ratio
- $e_1$  = Void ratio of unsaturated sample under pressure  $P_1$  for the conditions of no drainage; assumed to be instantaneous. Also, void ratio of saturated sample for conditions of full drainage under load  $P < P_1$ .
- $e_2$  = Ultimate void ratio under pressure  $P_1$
- $t$  = Time of consolidation
- $t_{100}$  = Time lapse for 100% consolidation under load  $P_1$
- $U$  = Pore pressure
- $U_{e_1}$  = Pore pressure due to volume change  $e_0 - e_1$  for conditions of no drainage. (Pore pressure formula, eq. 1)
- $V$  = Volume of escaped water during the consolidation process
- $\bar{\sigma}_n$  = Effective stress for volume change  $e_0 - e_1$

FIG. Y2.—STRESS, VOLUME CHANGE AND DRAINAGE RELATIONSHIP

where  $k$  is the coefficient of permeability,  $\gamma_w$  is the unit weight of water,  $m_v$  is the coefficient of volume change, and  $a_v = \frac{e_1 - e_2}{P_1 - \bar{\sigma}_{e1}}$  is the coefficient of compressibility for conditions of complete saturation.

As previously stated, the volume of water escaped from the unsaturated sample for the change of void ratio  $e_1 - e_2$  is  $V = (e_1 - e_2) + (e_0 - e_1)$ . Substituting

$$\begin{aligned} a_v(P_1 - \sigma_{e1}) &= e_1 - e_2; \quad V = a_v(P_1 - \bar{\sigma}_{e1}) + (e_0 - e_1) \\ &= (P_1 - \bar{\sigma}_{e1}) \left( a_v + \frac{e_0 - e_1}{P_1 - \bar{\sigma}_{e1}} \right) = (P_1 - \sigma_{e1}) a_v' \end{aligned}$$

where

$$a_v' = a_v + \frac{e_0 - e_1}{P_1 - \bar{\sigma}_{e1}} \quad \dots$$

The method was applied to predict pore pressure build-up at the location of piezometer #7 which is about 6 m above the ground surface. The missing data is the coefficient of consolidation  $c_v$  which was computed by the use of Eq. Y3. At first the value of  $k = 10$  ft per yr as presented by the author was used and it was found that complete consolidation will take place as fast as the load is placed. Although the coefficient of permeability reduces with the decrease of volume any speculations as to the value of  $c_v$  on the part of the writer will not contribute to a sound comparison between the observed and predicted pore pressure buildup. Therefore, if the author could possibly provide the values of  $c_v$  for consolidation tests which resemble the actual conditions such as presented in Figs. 6 and 7 of the original paper, the proposed method may be verified.

However, for the purpose of illustration let us assume that  $k = 10^{-6}$  cm per sec. The data needed for computations are presented in Figs. Y3 and Y4. The curve  $\bar{\sigma}$  versus  $\Delta$  as presented in Fig. Y3 is the actual curve recorded for settlement gage crossarms X 8-9 as presented in Figs. 6 and 7 of the original paper. It should be noted that the "as constructed" compression curve differs considerably from the laboratory  $\bar{\sigma}$  versus  $\Delta$  curve, presented in Fig. Y1.

The construction of the first 6 m of embankment was completed in about 45 days. Ultimate settlements under various loads are presented in Table Y1. Computations of the degree of consolidation at various construction stages is presented in Table Y2. The values of  $c_v'$  used in Table Y2 are presented in Table Y3.

The procedure employed to determine  $c_v'$  is as follows: Total load increments of 8.33 psi, representing 3 m of fill are used. The first total load increment is 8.33 psi, the vertical line intersects  $\bar{\sigma}$  versus  $\Delta$  curve at  $\Delta = 1.2\%$ , and the  $\sigma$  versus  $\Delta$  curve at  $\Delta = 0.83\%$ , which is the instantaneous compression for the total load of 8.33 psi.

For  $\Delta = 0.83\%$   $\bar{\sigma} = 5.8$  psi and  $U = 8.33 - 5.80 = 2.53$  psi. For unsaturated material the volume of water to leave the sample for the compression between 0.83% and 1.2% is 0.83% more than that of a saturated material in which the volume of water is  $1.20 - 0.83 = 0.37\%$ .

$$\text{By definition } a_v' = a_v + \frac{e_0 - e_1}{P_1 - \bar{\sigma}_{e1}}$$

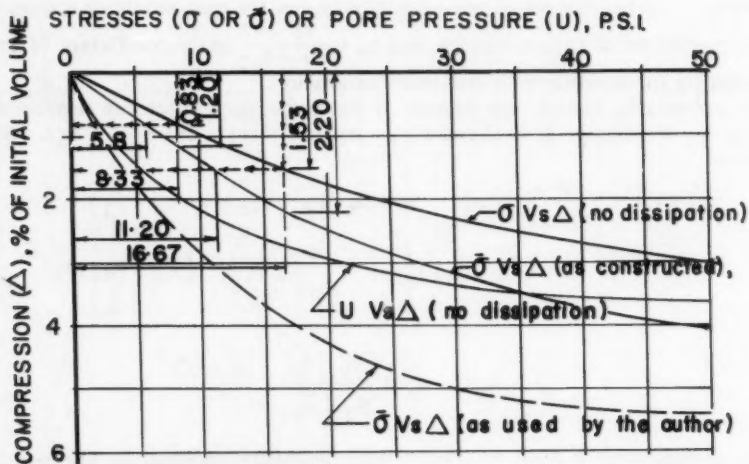


FIG. Y3.—COMPRESSION-STRESS RELATIONSHIP CURVES USED IN THE PROPOSED DESIGN METHOD

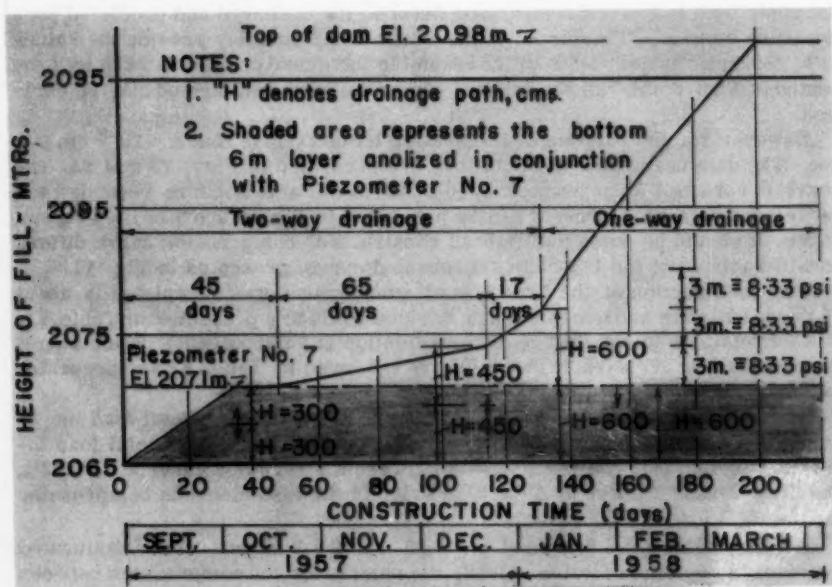


FIG. Y4.—"AS BUILT" - CONSTRUCTION SCHEDULE

TABLE Y1.—ULTIMATE SETTLEMENTS UNDER VARIOUS LOADS

Height of Fill, in cm	Load Increment	PSI, Cumulative	Layer Thickness, h in cm	Construction Time, in Days	Initial Void Ratio, $e_0$	$\Delta$	Ultimate Settlement in cm
300 <sup>a</sup>	8.33 <sup>a</sup>	8.33	600	45	0.7578	0.0120	7.20
300	8.33	16.67	600	65	0.7578	0.0220	13.20
300	8.33	25.00	600	17	0.7578	0.0280	16.80
300	8.33	33.33	600	18	0.7578	0.0335	20.10
300	8.33	41.67	600	10	0.7578	0.0375	22.50
300	8.33	50.00	600	10	0.7578	0.0410	24.60
300	8.33	58.33	600	12	0.7578	0.0433	25.98
300	8.33	66.67	600	12	0.7578	0.0450	27.00
300	8.33	75.00	600	10	0.7578	0.0460	27.60 <sup>b</sup>

<sup>a</sup> Taken as an average for the 45 construction days. <sup>b</sup> The sum of the measured settlement of crossarms X 7-8 and X 8-9 is 11.5 + 14.5 = 26.00. (Fig. 7 of the original paper.)

TABLE 2.—CONSOLIDATION OF THE BOTTOM 6-M LAYER OF EMBANKMENT ADJACENT TO PIEZOMETER NO. 7<sup>a</sup>

Load, in psi	Drainage Path, H wcm	Construction Time t		$C_v$ , in $\text{cm}^2/\text{min}$	$T_v$	Consolidation, in %	Ultimate Settlement of 6-M Layer in cm	Re-main-ing Possible Settlement, in cm	Actual Settlement for a Given Surcharge and Duration, in cm	
		Days	Min.						Incremental	Cumulative
8.33	300	45	64,800	0.91	0.655	85	7.20	1.07	6.13	6.13
16.67	450	65	93,600	1.04	0.479	75	13.20	6.07	4.56	10.69
25.00	600	17	24,480	1.38	0.094	35	16.80	6.11	2.14	12.83

Consolidation on Jan. 17, 1958 =  $\frac{12.83}{16.80} \times 100 = 76\%$ ; pore pressure =  $100 - 76 = 24\%$  of the surcharge load above the layer.

Surcharge Load =  $\frac{9 \times 123}{62.4} = 17.75$  m of water. Pore Pressure =  $0.24 \times 17.75 = 4.08$  m of water.

<sup>a</sup> For ultimate settlement see Table 1. Computations for other load ranges and duration are not shown.  $c_v$  =  $1.65 \text{ cm}^2$  per min assumed to be constant for all loads above 33.33 psi.

TABLE 3.—COMPUTED VALUES OF  $c_v^1$  FOR  $K = 6 \times 10^{-5}$  CM PER MIN<sup>a</sup>

Load, in psi	Compres- ion, in %		Effective Stress, in psi		$P_1 - \bar{\sigma}_{e1}$	$1 + e_0$	$e_1$	$1 + e_1$	$m_v^1$		$c_v^1$
	Instant	Ulti- mate	Instant	Ulti- mate					in <sup>2</sup> /lb	cm <sup>2</sup> /lb	
	$\Delta_1$	$\Delta_2$	$\bar{\sigma}_{e1}$	$P_1$							
8.33	0.83	1.20	5.80	8.33	2.53	1.7578	0.7432	1.7432	0.00478	0.0302	0.91
16.67	1.53	2.20	11.20	16.67	5.47	1.7578	0.7306	1.7306	0.00409	0.0263	1.04
25.00	2.10	2.80	15.75	25.00	9.25	1.7578	0.7205	1.7205	0.00308	0.0199	1.38
33.33	2.50	3.35	20.00	33.33	13.33	1.7578	0.7134	1.7134	0.00258	0.0166	1.65

<sup>a</sup> ALL  $c_v^1$  values for total loads in excess of 33.33 psi are assumed to be 1.65 cm<sup>2</sup> per min.

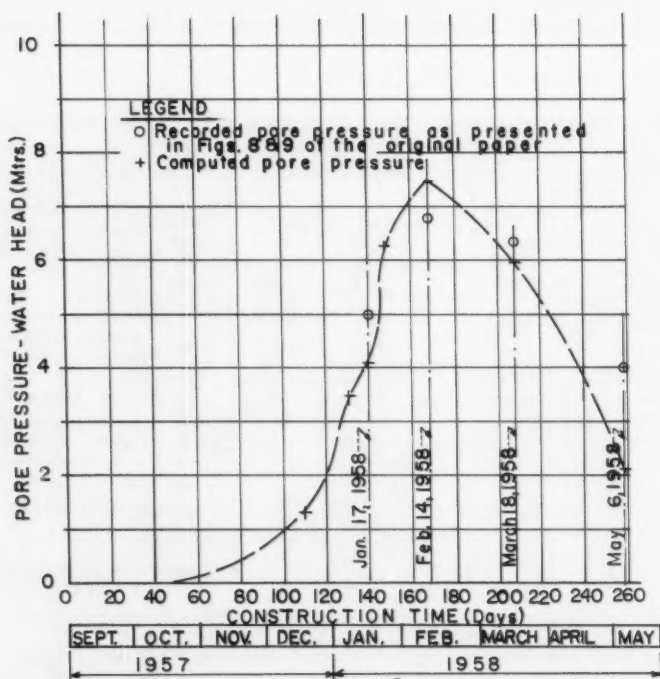


FIG. Y5.—PORE PRESSURE AT PIEZOMETER NO. 7  
 COMPARISON BETWEEN COMPUTED AND  
 RECORDED VALUES

From Eq. Y4

$$m_v' = \frac{a_v'}{1 + e_1} = \frac{e_1 - e_2}{(P_1 - \sigma_{e1})(1 + e_1)} + \frac{e_0 - e_1}{(P_1 - \bar{\sigma}_{e1})(1 + e_1)} = \frac{e_0 - e_2}{(P_1 - \bar{\sigma}_{e1})(1 + e_1)}$$

$$= \frac{e_0 - e_2}{(P_1 - \bar{\sigma}_{e1})(1 + e_1)} \frac{1 + e_0}{(1 + e_0)} = \frac{\Delta \bar{\sigma} = P_1}{(P_1 - \bar{\sigma}_{e1}) \times 100} \frac{1 + e_0}{(1 + e_1)}$$

where

$$\Delta = \frac{e_0 - e_2}{1 + e_0} \times 100 = \text{Change of volume, in \% , of initial total volume.}$$

Therefore,

$$m_v' = \frac{0.012}{2.53} \times \frac{1.7578}{1.7432} = 0.00478 \text{ in}^2 \text{ per lb} = 0.0302 \text{ cm}^2 \text{ per lb}$$

Substituting in Eq. Y3

$$C_v' = \frac{k}{\gamma_w m_v'} = \frac{6}{0.00219 \times 0.0302 \times 10^5} = 0.091 \text{ cm}^2 \text{ per min}$$

Computations for  $c_v'$  are presented in Table Y3. Comparison between predicted pore pressure at piezometer No. 7, computed by the writer's proposed method, and the recorded pore pressure is presented in Fig. Y5.

As previously mentioned, a comparison between computed and recorded results can be justified only if actual  $c_v'$  values are known.

In practice, design values are chosen by assuming most unfavorable pore pressure affecting conditions that may be encountered, such as the initial water content at placement, the minimum allowable density required by specifications and the rate of construction. If the suggested method is backed up by theory, then the part of the engineering judgment is confined only to the adopted design values, which is a common practice in the general civil engineering field.



## DESIGN OF UNDERSEEPAGE CONTROL MEASURES FOR DAMS AND LEVEES<sup>a</sup>

Discussions by H. R. Cedergren and Max Suter

H. R. CEDERGREN,<sup>1</sup> M. ASCE.—The authors are to be commended for their thorough treatment of the design of seepage control facilities for dams and levees. The many practical criteria they present should be of considerable benefit to engineers designing earth dams and levees on pervious foundations. Their paper demonstrates the importance of providing drainage facilities rather than depending on width of section for security against seepage.

When one reviews a paper of this kind the question comes to mind that if underseepage control measures are needed in new structures, "What about the levees that have been built without drainage facilities?" In the recent past and even at present some relatively important levees are being designed and built with little or no provision for seepage control. Those that have withstood the test of their design flood stages may have proved themselves; however, there are many miles of levee in existence that have never been subjected to the design flood stage, and, therefore, have not been fully tested.

An examination of the past performance of existing levees no doubt would, in some cases, indicate the wisdom of adding seepage control measures to provide increased protection. The principles outlined by the authors, and the flow net provide means for making such reviews.

Most of the control measures described by the authors lend themselves not only to new works, but also to the improvement of existing levees that may be found to need reinforcement.

As an example of the application of a seepage analysis to the design of control measures for improvement of an existing levee, the writer wishes to cite a drain design that was developed for a levee in the Pacific Northwest. This levee was part of a system that protected low-lying potential industrial land from flood stages of the Columbia River. The levee as originally constructed was of homogenous cross section, on a foundation of river sediments predominating in fine sand and silt. Heavy boils occurred at a number of points during an unusually high river stage; however, a breach in another part of the levee system flooded the land in back of the levee before the flood crest had been maintained an appreciable length of time.

In view of the potential value of the land in back of this levee, the owners decided that some additional protection against severe floods should be provided. To obtain the degree of security that was judged desirable the writer suggested a toe drain that would intercept seepage through the levee itself, and also provide reasonable protection against seepage through the foundation. The design is illustrated in Fig. C1. A trench is excavated well into the toe and

<sup>a</sup> October, 1959, by W. J. Turnbull and C. I. Mansur.

<sup>1</sup> Sr. Materials and Research Engr., Calif. Div. of Highways, Sacramento, Calif.

foundation during a low river stage, and a filter layer placed adjacent to the exposed levee slope and bottom of the trench. A small zone of filter gravel surrounds a collector pipe that is periodically connected to outfalls. To reduce cost, the filter blanket is relatively thin, and the bulk of the backfill is obtained from the material excavated for the drain. Flow-nets in Fig. C2 show theoretical seepage conditions for the simplified cross section before and after construction of the new drain. At this location the drainage trench shown could provide substantial removal of underseepage. Where the trench would be separated from deep pervious strata by soils of relatively low permeability, this drain would intercept only a small portion of the underseepage, however, it might, in such cases, be used in conjunction with relief wells.

The filter layers of drains, such as shown in Fig. C1, should be designed to prevent infiltration of the soil. Also, the layer in contact with the soil should be sufficiently more pervious than the soil in the levee and its foundation to remove seepage without appreciable buildup of head. The usual seepage analysis does not go beyond this point, and ignores the possibility of hydrostatic pressures building up within the filter itself. An analysis of this condition with

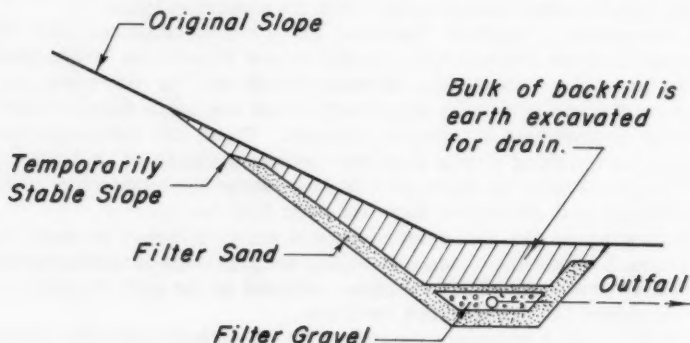


FIG. C1.—DETAILS OF TOE DRAIN DESIGN FOR IMPROVEMENT OF AN EXISTING LEVEE

the flow-net can throw light on the relative degree of permeability needed for free removal of seepage. A flow-net for the filter design illustrated in Fig. C1 is given in Fig. C3. The maximum depth of penetration of seepage into the sand layer is designated as  $T$ . Calculated values of  $T$  for several permeability ratios are tabulated in Fig. C3. For the conditions analyzed in Fig. C3, a filter layer with a permeability in the order of 10 times that of the soil would need to be only about one foot in thickness. For this kind of design, adequate permeability ordinarily is easily obtained; however, studies of this kind for a variety of engineering applications have convinced the writer that permeability requirements of filters in some common situations may be considerably greater than generally has been recognized. Simple studies of the type illustrated in Fig. C3 can provide reasonable permeability criteria for many practical drainage situations.

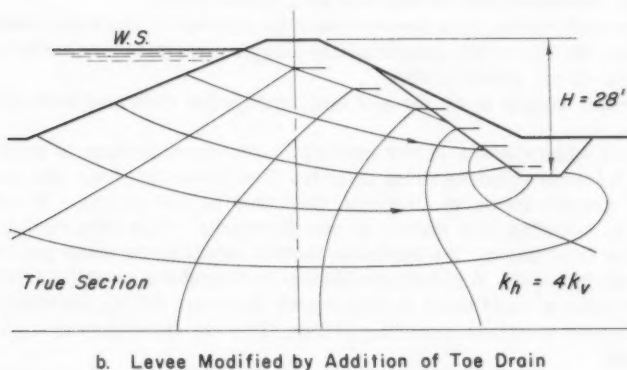
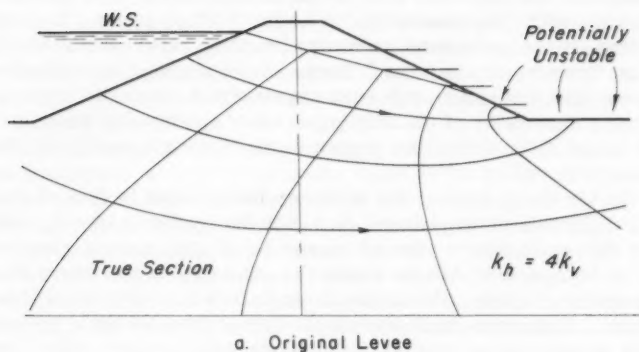


FIG. C2.—FLOW NETS SHOWING IMPROVEMENT OF A LEVEE  
ORIGINALLY BUILT WITH NO SEEPAGE  
CONTROL FACILITIES

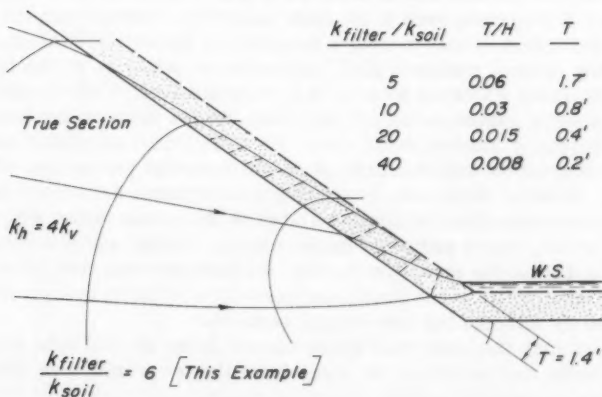


FIG. C3.—ENLARGED VIEW OF DISCHARGE FACE SHOWING FLOW-NET IN FILTER  
LAYER SLIGHTLY MORE PERVIOUS THAN SOIL

MAX SUTER,<sup>2</sup> F. ASCE.—This is the second paper in a series of three papers dealing with "underseepage." As in the first paper, this second one assumes that all the underseepage comes from the river, which is claimed to be based on "known seepage laws." These known seepage laws are formulated solely on visual examination, whereas physical and chemical tests mentioned in the writer's discussion of the first paper show clearly that most of the seepage water is not river water, but ground water flowing towards the river or in the alluvial river valley.

The criterion of any natural law is always that it shall be free of any contradiction. The "known seepage laws" on which this paper is based, certainly do not satisfy this criterion, in view of the results of physical and chemical tests. If, then, the fundamental law on which the calculations are based is not correct, the results of these calculations have to be weighed with extreme care.

The control measures shall prevent the danger of sand boils undermining a levee. This is hoped to be attained by methods which reduce either the underflow or the pressure beneath the land side of the levee.

The idea underlying this theory would be correct if the water would all be coming from the river, but unfortunately temperature measurements and water quality tests do not confirm this.

If the water comes from the hill side, somewhat different methods have to be applied.

Riverside blankets are fairly useless. The river bottom is mostly silted and this silt forms an impervious layer for flow from the river into the ground, but allows ground water to flow into the river at low stages. It acts like a check valve, allowing flow mainly in one direction. This may explain why, as stated in the first paper, the landside blanket tends to be more pervious than the riverside blanket. A riverside blanket is therefore mostly useless as far as the formation of sand boils is concerned. However, filling riverside borrow pits may reduce mosquito breeding places after the flood thus being helpful in this respect.

Sand boils occur where the weakest place is in the soil, and this is normally at the toe of the levee. Landside berms may shift this low point away from the toe of the levee and in this way reduce the danger that may exist to the toe of the levee from heavily sand producing sand boils. Such sand boils take the sand from the soil underneath their place of occurrence and thereby may undermine the toe of the levee, even if the main water flow comes from the land side.

The writer can see only negative benefits by relief wells at the bottom of the toe. Any control measure shall increase the security of the levee. Any seepage flow, from whatever source, is a nuisance against which other disposal measures, mostly pumpage, have to be taken. Relief wells at the toe of a levee form an artificially induced weak spot. No matter how carefully designed and built, they deteriorate and then can act as channels for the escape of sand during floods. Several cases are known where structures have been damaged by sand boils from abandoned wells. As stated in the paper, relief wells increase the flow of water, which certainly is no benefit. Relief wells are supposed to reduce the uplift on the levee, yet it may be remarked that any levee that cannot stand natural uplift is not stable and therefore of faulty design. This cannot be corrected by introducing new danger sources.

The theory that the sand boil water comes from the hill side may suggest that relief wells can be drilled far away from the levee and, thus, intercept the

<sup>2</sup> San Clemente, Calif.

flow, thereby reducing the heat at the toe of the levee and also the danger of forming sand boils there. Naturally this effect can, under favorable conditions, also be obtained by drainage ditches, drainage trenches or similar interceptors.

The fundamental principle of control is not the avoidance of the formation of sand boils, but to cause their formation to be located as far away from the toe of the levee as possible.

The correct measures require always careful local studies. Even then, failures can happen, as sand boils do not occur at the same place in every flood. Floods are not only different as to height and duration, but also in respect to local rainfall. In the upper reaches, high waters and heavy local rainfall coincide in general, unless the flood is due to melting snow, whereas in the lower reaches a flood may occur without local rainfall, and heavy local rainfall may have hardly any effect on river stages. Such variations have an effect on the frequency of occurrence of sand boils, as has been observed several times at Cairo, Ill.

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# PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

## VOLUME 85 (1959)

- APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998(SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007(HW2)<sup>c</sup>, 2008(EM2)<sup>c</sup>, 2009(ST4)<sup>c</sup>, 2010(SM2)<sup>c</sup>, 2011(SM2)<sup>c</sup>, 2012(HY4)<sup>c</sup>, 2013(PO2)<sup>c</sup>.
- MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(SA3), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)<sup>c</sup>, 2041(PL2)<sup>c</sup>, 2042(PP1)<sup>c</sup>, 2043(ST5)<sup>c</sup>, 2044(SA3)<sup>c</sup>, 2045(HY5)<sup>c</sup>, 2046(PP1), 2047(PP1).
- JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056(HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065(ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)<sup>c</sup>, 2073(IR2)<sup>c</sup>, 2074(PO3)<sup>c</sup>, 2075(ST6)<sup>c</sup>, 2076(HY6)<sup>c</sup>, 2077(SM3)<sup>c</sup>, 2078(WW2)<sup>c</sup>.
- JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087(SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096(EM3), 2097(HY7)<sup>c</sup>, 2098(SA4)<sup>c</sup>, 2099(EM3)<sup>c</sup>, 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).
- AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134(SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)<sup>c</sup>, 2139(PO4)<sup>c</sup>, 2140(SM4)<sup>c</sup>.
- SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)<sup>c</sup>, 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158(IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176(WW3), 2177(WW3), 2178(CO2)<sup>c</sup>, 2179(IR3)<sup>c</sup>, 2180(HW3)<sup>c</sup>, 2181(SA5)<sup>c</sup>, 2182(HY9)<sup>c</sup>, 2183(SU1)<sup>c</sup>, 2184(WW3)<sup>c</sup>, 2185(PP2)<sup>c</sup>, 2186(ST7)<sup>c</sup>, 2187(PP2), 2188(PP2).
- OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO6), 2207(PO6), 2208(PO6), 2209(PO6), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO6), 2227(PO6), 2228(PO6), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)<sup>c</sup>, 2233(PL3)<sup>c</sup>, 2234(EM4)<sup>c</sup>, 2235(HY10)<sup>c</sup>, 2236(SM5)<sup>c</sup>, 2237(ST8)<sup>c</sup>, 2238(PO6)<sup>c</sup>, 2239(ST8), 2240(PL3).
- NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)<sup>c</sup>, 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)<sup>c</sup>, 2270(ST9).
- DECEMBER: 2271(HY12)<sup>c</sup>, 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)<sup>c</sup>, 2324(CP2)<sup>c</sup>, 2325(SM6)<sup>c</sup>, 2326(WW4)<sup>c</sup>, 2327(IR4)<sup>c</sup>, 2328(PO6)<sup>c</sup>, 2329(ST10)<sup>c</sup>, 2330(CP2).

## VOLUME 86 (1960)

- JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)<sup>c</sup>, 2349(HY1)<sup>c</sup>, 2350(ST1), 2351(ST1), 2352(SA1)<sup>c</sup>, 2353(ST1)<sup>c</sup>, 2354(ST1).
- FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)<sup>c</sup>, 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)<sup>c</sup>, 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)<sup>c</sup>, 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)<sup>c</sup>, 2391(SM1)<sup>c</sup>, 2392(PO1)<sup>c</sup>.
- MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)<sup>c</sup>, 2419(WW1)<sup>c</sup>, 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)<sup>c</sup>, 2426(HY3)<sup>c</sup>, 2427(ST3)<sup>c</sup>.
- APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436(ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445(EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454(EM2), 2455(EM2)<sup>c</sup>, 2456(HY4)<sup>c</sup>, 2457(PO2)<sup>c</sup>, 2458(ST4)<sup>c</sup>, 2459(SM2)<sup>c</sup>.

c. Discussion of several papers, grouped by divisions.

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# PART 2

APRIL 1960 — 14

VOLUME 86

NO. SM 2

PART 2

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**NEWS  
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JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION  
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS



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## DIVISION ACTIVITIES

### SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

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#### NEWS

April, 1960

#### NEWS FROM THE WEST COAST

##### Meetings

Two recent meetings have been held by the Soil Mechanics and Foundations Division of the San Francisco Section. On the 9th of December the technical program consisted of a panel discussion on subdivision soil problems. The panel members and their subjects were:

Lester A. Phebus, F.H.A. Site Engineering and Acting Chief Land Planner  
Carl Timmons, Subdivision Engineer, Westlake

"What is expected of a Soil Engineer in the Development of a Subdivision"

Arnold Olitt, Soil Engineer, Woodward, Clyde, Sherard and Associates

"What a Soil Engineer Can Do in the Development of a Subdivision"

The program consisted of short talks which were followed by a lively question and answer period with audience participation. The subject was a popular one in California where a population explosion is being experienced. On January 28, 1960 Professor Harry Seed of the University of California presented a talk on the subject of "Soils and Foundation Work in the USSR." Professor Seed, who was part of an exchange group which recently returned from Russia, presented an excellent analysis of current work in Russia illustrated by color slides.

At a joint meeting of the Hydraulics and Power Division of the San Francisco Section, held 19 January, J. Barry Cooke of the Pacific Gas and Electric Company presented a talk on the failure of the Malpasset Dam in France. Barry had an opportunity to visit the site of the failure in southern France and to speak with engineers and local observers on the design, construction and failure of thin thin arch dam which suddenly failed and caused the death of some 400 persons. The talk was most interestingly presented and illustrated with numerous slides to an overflow audience of some 225.

At a regular meeting of the San Francisco Section on February 16, Kenneth Hoover, Chief Engineer of the Bay Area Rapid Transit District presented a talk "Rail Rapid Transit—an Engineering Report." One of the most interesting features of this project includes studies now being made on the feasibility

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of construction of a rapid transit tube across San Francisco Bay from San Francisco to Oakland. Currently a soils exploration program is under way along the alignment of the proposed \$115,000,000 underwater rapid transit tube, a four-mile long link in the overall regional transit system. In addition to conventional underwater soil sampling, geophones are being placed in the completed borings to monitor earthquake tremors along the tube alignment. These instruments—attached by telephone to recorders in a downtown San Francisco office—will remain in operation until tube construction begins in an estimated two years.

#### NINTH NATIONAL CLAY CONFERENCE

The Ninth National Clay Conference, sponsored by the Clay Minerals Committee of the National Academy of Sciences-National Research Council, will be held at Purdue University, Lafayette, Indiana, on October 6, 7 and 8, 1960. In addition, a short field trip and laboratory visits are planned for Wednesday, October 5. All those interested in research or technology in fields related to clays or clay minerals are cordially invited to participate.

#### Program

Two special symposia of invitational papers are planned on the subjects of "Engineering Aspects of Physico-Chemical Properties of Clays" and "Clay-Organic Complexes." The symposium on "Engineering Aspects of Physico-Chemical Properties of Clays" will feature contributions on soil mechanics. The "Clay-Organic Complexes" symposium will be concerned with basic studies of clay-organic systems in the paper and petroleum industries, in soil science, civil engineering, and other areas. In addition, there will be general sessions and original contributions on any aspect of clay mineralogy are invited.

Papers presented at the conference may be published in the Proceedings of the Conference (Clays and Clay Minerals), if accepted by the Board of Editors. The proceedings will be published by Pergamon Press.

Titles and abstracts of contributions to the general sessions should be submitted to Dr. J. L. White, Agronomy Department, Purdue University, Lafayette, Indiana, by June 1, 1960. Abstracts should be limited to 250 words and should be informative rather than descriptive. Abstracts should be submitted in duplicate.

#### Field Trip

A short field excursion is planned in the immediate vicinity of Lafayette to examine well-characterized soil profiles and geological features of the area.

#### Laboratory Visits

Guided tours through the Purdue University Soil Mechanics and Highway Research Project Laboratories as well as the Soil Chemistry and Soil Mineralogy Laboratories of the Agronomy Department are planned for Wednesday, October 5.

## Housing

The Union Club of the Purdue Memorial Union has adequate accommodations for all attending the Conference. Single rooms are \$5.00 and up; double rooms are \$7.00 and up; there are dormitory-type facilities for up to 36 persons at \$2.50 per person.

Additional accommodations are available at the Van Orman-Fowler Hotel, the Morris Bryant Motel, Cedar Crest Motel, Esquire Motel and Howard Johnson's Motor Lodge, Lafayette, Indiana. Reservations for these facilities should be mentioned in your reservation.

For further details write: Dr. J. L. White, Local Chairman  
Ninth National Clay Conference  
Agronomy Department  
Purdue University  
Lafayette, Indiana

## BRITISH CONFERENCE ON PORE PRESSURE AND SUCTION IN SOILS

### Object of the Conference

The discussions at the Fourth International Conference on Soil Mechanics and Foundation Engineering held in London in 1957 indicated a growing awareness of the importance of both negative and positive pore pressure in the whole field of soil mechanics. Since that Conference there is every indication that the process has continued.

To facilitate an interchange of ideas before the next international meeting in Paris 1961, the British National Committee has arranged a three-day conference on pore pressure and suction in soils.

### Place and Date of the Conference

By courtesy of the Council of the Institution of Civil Engineers the Conference will be held at the Institution of Civil Engineers, Great George Street, Westminster, London, S.W. 1, from 30 March to 1 April 1960, inclusive.

### Participants

The Conference is arranged for the benefit of members of the British National Society. Non-members may attend by invitation of the British National Committee.

### Papers

The following is a list of the papers to be discussed at the Conference:

- A. D. M. Penman: "A study of the response of time of various types of Piezometer."
- K. L. Nash and R. K. Dixon: "The measurement of pore pressure in sand under rapid triaxial test."
- A. W. Bishop: "The measurement of pore pressures in triaxial test."
- R. Peltier: "Les recherches sur la succion des sols au Laboratoire Central des Ponts et Chaussees."
- G. D. Aitchison: "Relationship of moisture stress and effective stress functions in unsaturated soils."

- A. L. Little and A. J. Vail: "Some developments in the measurement of pore pressure."
- J. Paton and N. G. Semple: "Investigation of the stability of an earth dam subject to rapid drawdown, including details of pore pressures recorded during a controlled drawdown test."
- R. E. Gibson and A. Marsland: "The measurement of pore pressures and settlements under a loaded oil tank."
- A. W. Bishop, A. D. M. Penman, and M. F. Kennard: "Pore pressure observations at Selset Dam."
- J. R. Lake: "Pore pressure measurements in model-scale and full-scale experiments to determine the effectiveness of vertical sand drains in peat subsoils."
- A. W. Skempton and D. J. Henkel: "Field observations on pore pressure in London Clay."
- E. C. W. A. Geuze: "Pore pressure measurements in the laboratory and in the field."
- L. Bjerrum and I. Johannessen: "Pore pressures resulting from driving piles in soft clay."
- D. Croney and J. D. Coleman: "Pore pressure and suction in soil."
- J. E. Jennings: "A revised effective stress law for use in the prediction of the behaviour of unsaturated soils."
- P. Habib: "Contribution a la mesure des pressions interstitielles."
- W. R. Gardner: "Soil suction and water movement."
- R. K. Schofield: "Suction in swollen clays."

#### Proceedings

After the Conference, copies of the Proceedings will be obtainable from:

Butterworths Scientific Publications  
4-5 Bell Yard  
Temple Bar, London, W.C. 2.

Approximate price L2 per copy.

#### RUSSIAN JOURNAL OF FOUNDATIONS AND SOIL MECHANICS

VOL. 1, NO. 4

OSNOVANIA, FUNDAMENTY I MEKHANIKA GRUNTOV (FOUNDATIONS AND SOIL MECHANICS). Issued by Gosudarstvennyi komitet Soveta Ministrov SSSR po delam stroitel'stva (State Committee of the Council of Ministers of the U.S.S.R. for Construction). Moscow, no. 4, 1959.

Table of contents translated:

- Barkan, D. D.—Principal problems in the further development of vibration methods in construction, p. 1.
- Egorov, K. E.—Studying deformations in foundation layers of flues, p. 4.
- Goncharov, I. U. M.—Using the Coulomb theory in determining soil pressure on elastic walls, p. 7.
- Gol'dshtein, M. N., Babitskaia, S. S.—Methods for determining the long-time strength of soils, p. 11.
- Molotilov, B. V.—Some remarks on the Standards and Technical Specifications 127-55, p. 15.

- Mikheev, V. V.—Practical problems in using standards and technical specifications in designing natural foundations of buildings and industrial structures (NiTU 127-55), p. 16.
- Ushkalov, V. P.—Compressibility of thawing foundation soils according to field-test data, p. 19.
- Druzhinin, M. K., Gorelik, A. M.—Depth of foundations to be laid on heaving soils, p. 22.
- Gorlovskii, B. L.—Building on slag and cinder fills, p. 25.
- Rudnitskii, N. IA.—Collapsing of ceilings of unfinished buildings caused by frost action under foundations, p. 27.
- Rozhdestvenskii, N. A., Trofimenkov, IU. G.—Constructing deep foundations in Yugoslavia, p. 28.
- Information, p. 30.
- Reviews and bibliography, p. 31.

If sufficient interest is expressed, means of making some or all of the articles available in English will be explored.

#### ASCE RESEARCH CONFERENCE ON SHEAR STRENGTH OF COHESIVE SOILS

Plans for the conference at the University of Colorado, Boulder, Colorado, June 13-17 are nearing completion. The following tentative program of sessions and other activities is announced. In addition, an interesting program for ladies is planned.

Sunday, June 12, 1960

Arrival and registration

Monday, June 13, 1960

A. M. Arrival and registration

P. M. Session 1: Formal opening addresses followed by presentation and discussion of a paper on "Failure Hypotheses" by Nathan Nemark.

Evening. Get-acquainted hour at the hotel, Harvest House, followed by informal dinner and dancing

Tuesday, June 14, 1960

A.M. Session 2: Panel discussion of testing equipment, techniques, and errors; moderator, Arthur Casagrande

P.M. Session 3: Panel discussion of the shear strength of saturated clays; moderator, Stanley J. Johnson

Evening. Barbecue and square dancing at Memorial Center Terrace, University of Colorado

Wednesday, June 15, 1960

A.M. Session 4: Panel discussion of the shear strength of undisturbed cohesive soils; moderator, Ralph B. Peck

P.M. Excursions to Colorado-Big Thompson Project and U. S. Air Force Academy

Thursday, June 16, 1960

A.M. Session 5: Panel discussion of the shear strength of compacted cohesive soils; moderator, H. Bolton Seed

P.M. Session 6: Panel discussion of problems associated with the practical application of shear strength data; moderator, Philip C. Rutledge

Evening. Banquet at Harvest House; principal speaker, Ellis L. Armstrong, U. S. Commissioner of Public Roads

Friday, June 17, 1960

A.M. Session 7: Moderators' reports, general discussion, and closing addresses

P.M. Open house at the Bureau of Reclamation laboratories

In addition to the moderators listed above, associate moderators and panelists are being chosen by the Task Committee to provide a stimulating and instructive series of panel discussions with opportunities for questions and discussions from the floor. Panelists will include distinguished foreign as well as American soils engineers, who will base their discussions on a series of papers which will be preprinted and distributed to registrants in advance of the Conference. These papers will represent the views and practices of noted organizations and authorities in soil mechanics.

All those interested in shear strength of soils are urged to attend. Because of the ideal vacation facilities available in Colorado, the participants are encouraged to bring their families to the Conference. The University of Colorado will place a residence hall, with comfortable rooms and dining-room facilities, at the disposal of the Conference. The rates, including three meals per day, are \$42 for single-occupancy rooms and \$32 per person for double-occupancy rooms, covering a period starting Sunday night, June 12, and ending with breakfast on Saturday morning, June 18, 1960. The conference hotel is the Harvest House of Boulder, a new luxury-type hotel, with rates of \$9 a day for single-bed rooms and \$12 a day for twin-bed rooms without meals. Several older and smaller hotels and many new and comfortable motels are located in or near Boulder.

The Conference is sponsored by the Soil Mechanics and Foundations Division, ASCE. The Colorado Section, ASCE, and the University of Colorado will be hosts to the Conference. Dr. W. J. Turnbull is Chairman of the Task Committee on Shear Strength of Soils, charged with the general organization of the Conference, and Mr. W. G. Holtz is Chairman of the Local Committee on Arrangements.

Anyone interested in attending the Conference or in obtaining a copy of the proceedings is urged as soon as possible to fill in the following pre-registration coupon or the one attached to the official brochure announcing the Conference. Preregistration will greatly facilitate the work of the organizing committees and will assure printing of an adequate number of copies of the proceedings.

#### JUNE NEWSLETTER

Deadline date for arrival at this office of contributions for the June Newsletter: April 20, please.

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#### PREREGISTRATION COUPON

I plan to attend the Research  
Conference on Shear Strength  
of Cohesive Soils in Boulder,  
Colorado, June 13-17, 1960.

I inclose \$15\* which covers  
registration fee and entitles  
me to all preprints and a copy  
of the final Conference Pro-  
ceedings.

I may attend the Conference  
and shall inform the Secretary  
when my plans are definite.

(I shall want a copy of the  
Proceedings.)

I do not plan to attend the  
Conference, but I am interested  
in purchasing a copy of the  
Conference Proceedings at  
approximately \$10 to ASCE  
members and double this  
amount to nonmembers.

My wife and \_\_\_\_\_ children  
(ages \_\_\_\_\_) will  
accompany me.

Please send me Chamber of  
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I am interested in the following  
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Address \_\_\_\_\_

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\* Make checks payable to:

ASCE Shear Conference; address envelope to:

Dr. J. W. Hilf, Secretary

Task Committee on Shear Strength of Soils, ASCE

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